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GUIDELINES FOR EVALUATING THE SEISMIC RESISTANCE OF EXISTING BU--ETC(U)
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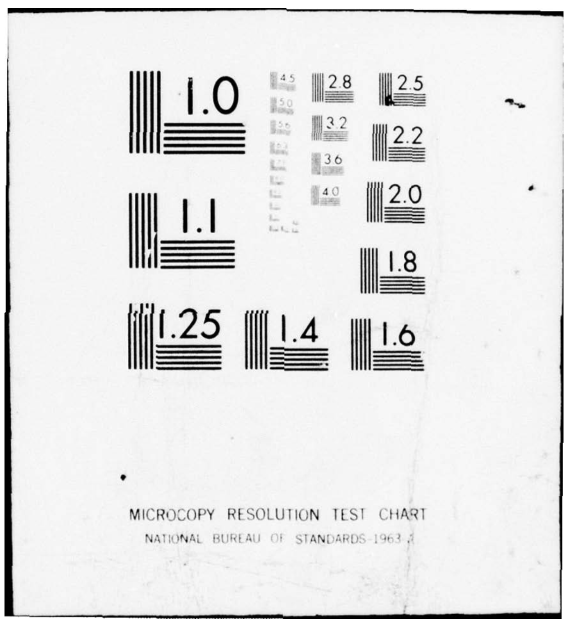
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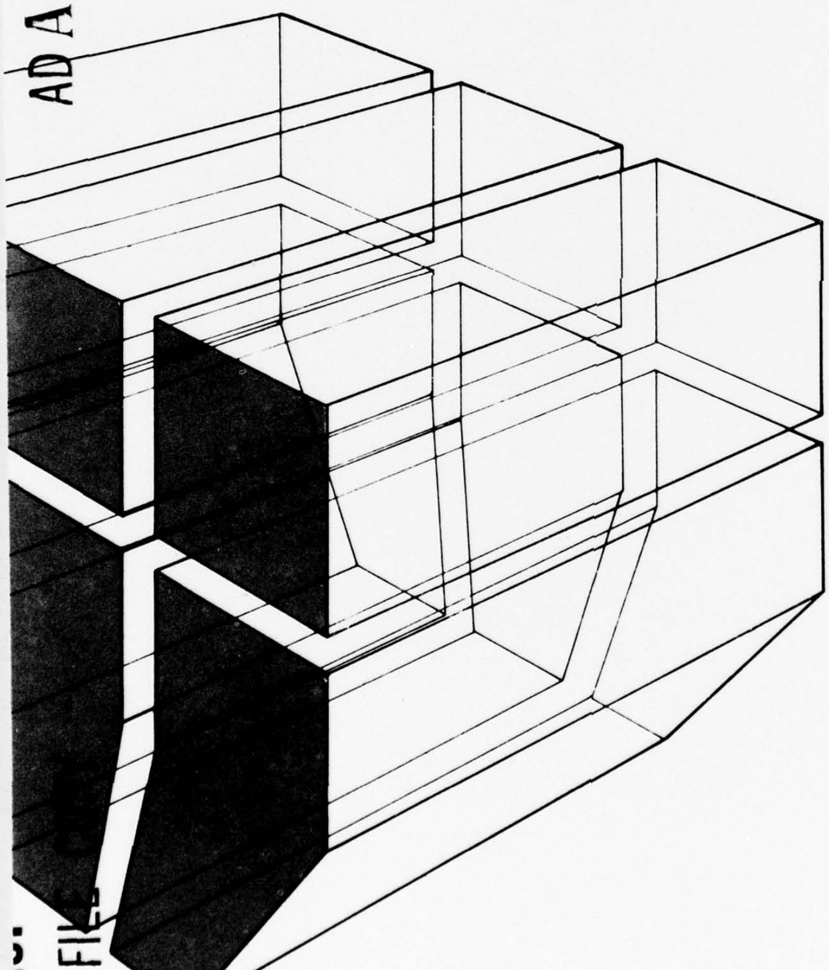
TECHNICAL REPORT M-213
July 1977
Design Guide for Seismic Strengthening of Existing Facilities

GUIDELINES FOR EVALUATING THE SEISMIC RESISTANCE OF EXISTING BUILDINGS

by
John M. Lybas



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This report presents a methodology for evaluating an existing building's seismic resistance. The method uses a design response spectrum and modal analysis techniques to compute a linearly elastic structural response. This is compared to the structure's yield capacity to obtain a factor denoting the structure's required energy dissipation capacity. Charts suitable for design office application are provided for use in estimating the levels of structural damping and ductility consistent with the required energy dissipation. Various structural characteristics pertinent to the building's suitability for seismic

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loading are discussed. The method compares these structural characteristics to the computed structural damping and ductility to reach a judgment concerning the likelihood of structural failure during an earthquake. Several appendices are included to provide additional technical support for the methodology.

FOREWORD

This project was conducted by the Structural Mechanics Branch (MSS), Materials and Science Division (MS), U.S. Army Construction Engineering Research Laboratory (CERL) for the Directorate of Military Construction, Office of the Chief of Engineers (OCE). The work was funded under RDT&E Army Program 6.27.19A, Project 4A762719AT41, "Design, Construction and Operation and Maintenance Technology for Military Facilities"; Task T4, "Military Construction Technology"; Work Unit 003, "Design Guide for Seismic Strengthening of Existing Facilities." The applicable QCR number is 1.03.003. The OCE Technical Monitor was Mr. G. M. Matsumura, Engineering Division.

COL J. E. Hays is Commander and Director of CERL, and Dr. L. R. Shaffer is Technical Director. Dr. G. R. Williamson is Chief of MS, and Dr. W. E. Fisher is Chief of MSS.

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1 INTRODUCTION

Background

The design and construction of U.S. Army facilities are based on the *Department of Defense Construction Criteria Manual* (DOD 4270.1)[1] and Army Technical Manuals (TMs), including the codes referenced therein. The seismic design load requirements are generally increased with each design code revision. Based on the results of recent earthquake damage, existing facilities usually have inadequate seismic structural resistance relative to the current requirements for new construction. Unless these facilities are reevaluated--especially those built prior to adoption of seismic codes or under early seismic codes--the potential for life hazard, economic losses, and possible loss of military function during seismic disturbance cannot be fully determined.

Purpose

The purpose of this report is to provide the basis for a rational approach to evaluating the seismic resistance of existing structures with reinforced concrete and masonry construction.

Approach

This evaluation method explicitly considers both the dynamic properties of the building and its inelastic response during seismic activity. It allows for consideration of the inherent advantages and disadvantages of the various types and arrangements of structural systems. The method allows flexibility with respect to the depth of evaluation; i.e., the extensiveness of the work can be tailored to accommodate the complexity and function of a particular building.

Scope

The procedure described here is considered an approximate concept. This is based on (1) the necessity that the procedure be practical for design office use and (2) the fact that the dynamic, inelastic

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[1] *Department of Defense Construction Criteria Manual*, DOD 4270.1 (Department of Defense, 1972).

structural response that characterizes the state of the art of earthquake engineering is not fully understood. The format of the method presented herein is such that the structural behavior of the members during seismic activity can be easily understood and efficiently incorporated into the method. Basically, what is provided is an evaluation guide. The method is an aid to obtaining technical data which will facilitate prudent judgment concerning the capability of a building to resist seismic loads.

Organization of Report

This report is divided into two major parts. The first (Chapters 2 through 9) describes the evaluation method in a concise, step-by-step fashion for use in the design office. The second is a set of seven appendices which provide a more detailed technical background and explanation of the evaluation procedure.

Chapters 2 through 9 detail the considerations necessary in the evaluation procedure, which is outlined in Figure 1. The procedure first involves a general observation of the building's characteristics. Consideration is given to structural details such as forces in joints, shear in flexural members, and reinforcement characteristics. These observations can be the basis of decisions concerning how detailed an evaluation to perform and whether to perform a simplified analysis in which a linear first-mode shape is assumed. At any rate, through use of a design response spectrum, along with the computed modal parameters, a base shear consistent with elastic response can be computed. This is compared to a base shear corresponding to a collapse mechanism. From this comparison, a parameter characterizing the required level of energy dissipation in the structure can be computed. From this energy dissipation factor, the required damping and ductility for the structural response can be inferred. This result is considered--along with the approximate damping and ductility capacities of the structure, the general observations mentioned previously, and various nonstructural characteristics--to arrive at a decision concerning the seismic hazard posed by the building.

Appendix A provides background for the general concept underlying the evaluation procedure--its use of an energy dissipation factor related to structural damping and ductility. Appendix B provides technical background relative to the evaluation of structural details. Appendix C contains additional guidance relative to the structural analysis portion of the evaluation. Appendix D contains some brief comments concerning the building's nonstructural and architectural features. Appendix E provides a short explanation of torsional behavior in a building. Appendix F explains the nature of the approximations involved in the method, and Appendix G defines the symbols used in the report.

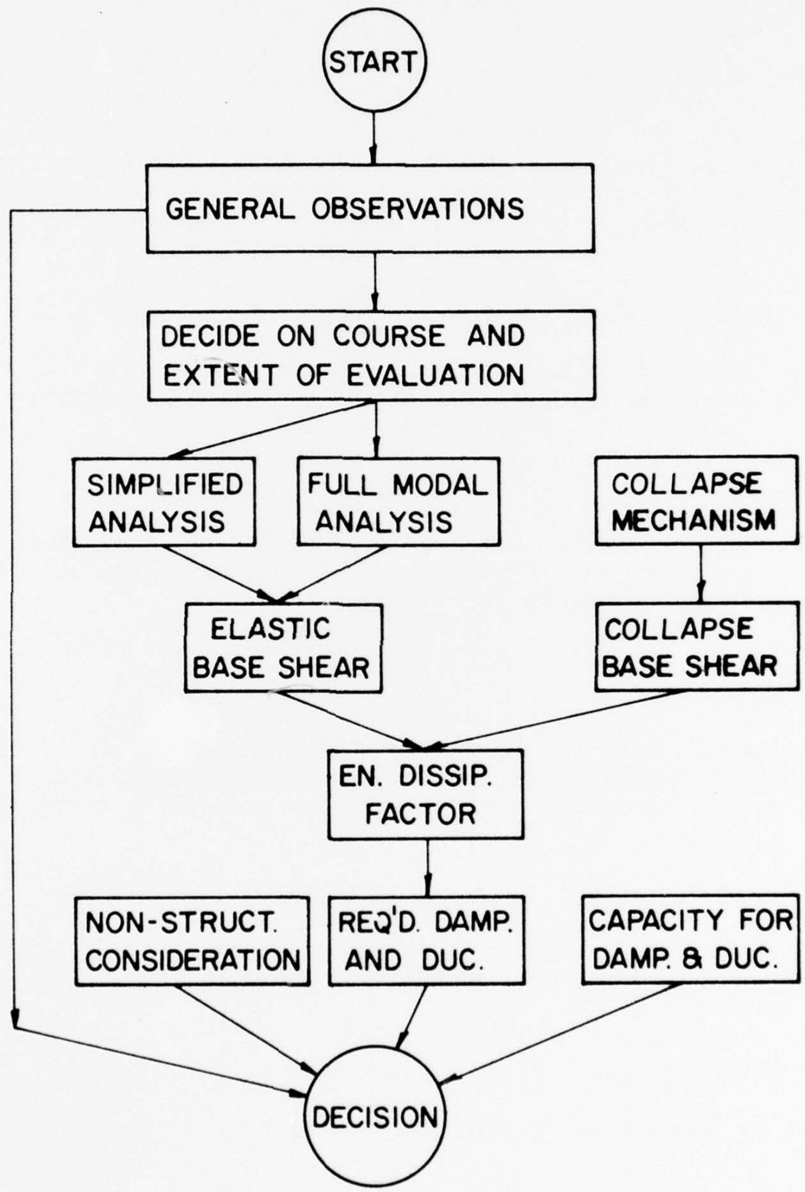

Figure 1. Outline of evaluation method.

Mode of Technology Transfer

This study will impact TM 5-809-10[2] through incorporation of parts of the evaluation procedure in that manual.

2 GENERAL CONSIDERATIONS

Several of the building characteristics that greatly affect seismic behavior can be investigated merely through observation or simple calculations. These characteristics are discussed in this chapter and summarized in Table 1.

General Characteristics

The first step in the evaluation is to note the general characteristics of the building discussed in the following paragraphs.

Functions

The functions of the structure, including its importance to the operation of the overall facility and the economic consequences of its failure must be considered.

Plans

"As-built" drawings and design analysis of the facility should be obtained. In the many cases where drawings and design analysis are not available, the evaluation must be based on prevailing criteria.

Variations From Plans

The engineer should check for any possible variations from the as-built drawings, especially additions or modifications.

Visible Damage

The building must be inspected for damage, since any damage may

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[2] *Seismic Design for Buildings*, TM 5-809-10 (Department of the Army, April 1973). This publication is commonly called the Tri-Services Manual as it is jointly published by the Army, Navy (as NAVFAC P-355), and Air Force (as Chapter 13 of AFM 88-3).

Table 1

Sources of Undesirable Seismic Behavior for Various Structural Systems

Structural Type	Problem Sources
All Types	1. Highly irregular plan 2. Changes in layout with height 3. Large eccentricity between center of mass and center of stiffness 4. Flexible first story 5. Setbacks or attached towers 6. Attached structures 7. Flexible or deteriorated diaphragms
Masonry Walls	1. Deteriorated condition--either in joinery or masonry units 2. Very low steel ratio or lack of steel 3. Poor grouting of reinforcement 4. Lack of reinforcement concentration around openings
Reinforced Concrete Walls	1. Low shear strength compared to flexural strength 2. Lack of boundary elements 3. Lack of reinforcement concentration around openings
Reinforced Concrete Frames	1. Lack of shear strength compared to flexural strength in beams and columns 2. Lack of confinement for core concrete in columns 3. Lack of shear strength in beam-column or slab-column joints 4. Lack of anchorage for main flexural steel 5. Lack of confinement for concrete in joints 6. Tendency to form flexural hinges in columns, rather than beams
Precast Concrete	1. Poor joints between precast elements

significantly increase its hazard potential during seismic loading. Damage includes cracking or spalling of concrete, corrosion of concrete or steel, and loose ornamentation or other loose architectural features, such as panels, veneers, or tiles. The damage may have been caused by previous earthquakes, weather, vibration, or fatigue from machinery, accidents, erection problems, or other factors. The condition of construction joints must also be examined. The engineer must decide what corrective action must be taken for the observed damage. Strengths or stiffnesses of members must be reduced if damaged conditions are severe. If key lateral load-resisting elements are severely damaged, the building must be repaired. The course of action to be taken may be determined based on the evaluation at this point.

Fire History

Records must be checked to determine whether the building has ever been subjected to a major fire. Fire will reduce concrete compressive strength and the bond strength between steel and concrete.[3] (Further effects of fire on structural members are discussed in "Effects of Elevated Temperature on Structural Members."[4]) The effects of fire may not be readily apparent, since they may have been hidden by paint or plaster without restoration of the structural integrity. A judgment is required about the extent of the damage to the structural members. For the purpose of calculations relating to evaluation, material strengths or section properties may be reduced or repairs may be made.

Earthquake History

Another class of damage, often concealed by later painting or plastering, is that caused by previous earthquakes. The nature of damage may be ascertained from written records or old photographs. For these cases, the engineer must decide how much this damage will affect the building's resistance to future seismic activity.

Material Properties

In cases of old or very important buildings, testing samples of the structure's concrete or masonry may be necessary. In some cases, it may be possible to remove samples of masonry or concrete from the structure and test them for shear and compressive strength. Several

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[3] Architectural Institute of Japan, *Design Essentials in Earthquake Resistant Buildings* (Elsevier Publishing Company, 1970).
[4] T. Uddin and C. G. Culver, "Effects of Elevated Temperature on Structural Members," *ST7, ASCE Proceedings* (July 1975).

sizes of specimens must be tested, since this may affect the strength.[5] For cases in which the removal of a sufficient number of samples for testing would seriously damage the structural system, there are several methods and devices available for in-place evaluation of concrete or masonry.[6]

Layout of Structure

The general idealization of the three-dimensional structural system is shown in Figure 2. The structure is idealized as a number of lateral load-resisting elements, including frames, structural walls, or combinations thereof, connected by diaphragms representing the floors and roof. In most cases, the structure's weight is considered to be concentrated at the floor levels; i.e., the total mass of the structure is assumed to exist in the diaphragms. (At times, however, this may not be a good assumption for very thick masonry walls, and the engineer's discretion is required.) In the concentrated mass idealization, the inertial forces associated with the earthquake are applied through the diaphragms' centers of mass and are distributed to the vertical lateral load-resisting elements. The nature of the force distribution will be a function of the arrangement and stiffnesses of the vertical elements and the stiffnesses of the diaphragms. Cases of highly irregular or nonuniform vertical elements, flexible diaphragms, or bizarre configurations in the plan usually lead to unusual force distributions throughout the structural system. The resulting stress concentrations tend to cause poor behavior of the structural system under seismic loading. Additionally, such concentrations are often difficult to predict in an analysis. The damage to the Veterans Administration Olive View Hospital complex during the 1971 San Fernando Valley earthquake is a prime example of problems caused by variations in lateral load-resisting elements from

[5] S. G. Fattal and L. E. Cattaneo, "Evaluation of Structural Properties of Masonry in Existing Buildings," *Earthquake Resistant Design Requirements for VA Hospital Facilities*, Report of the Earthquake and Wind Forces Committee (Veterans Administration, Office of Construction, March 1975).

[6] M. V. Maholtra, "In-Place Evaluation of Concrete," *ASCE Journal of the Construction Division*, Vol 101 (June 1975).

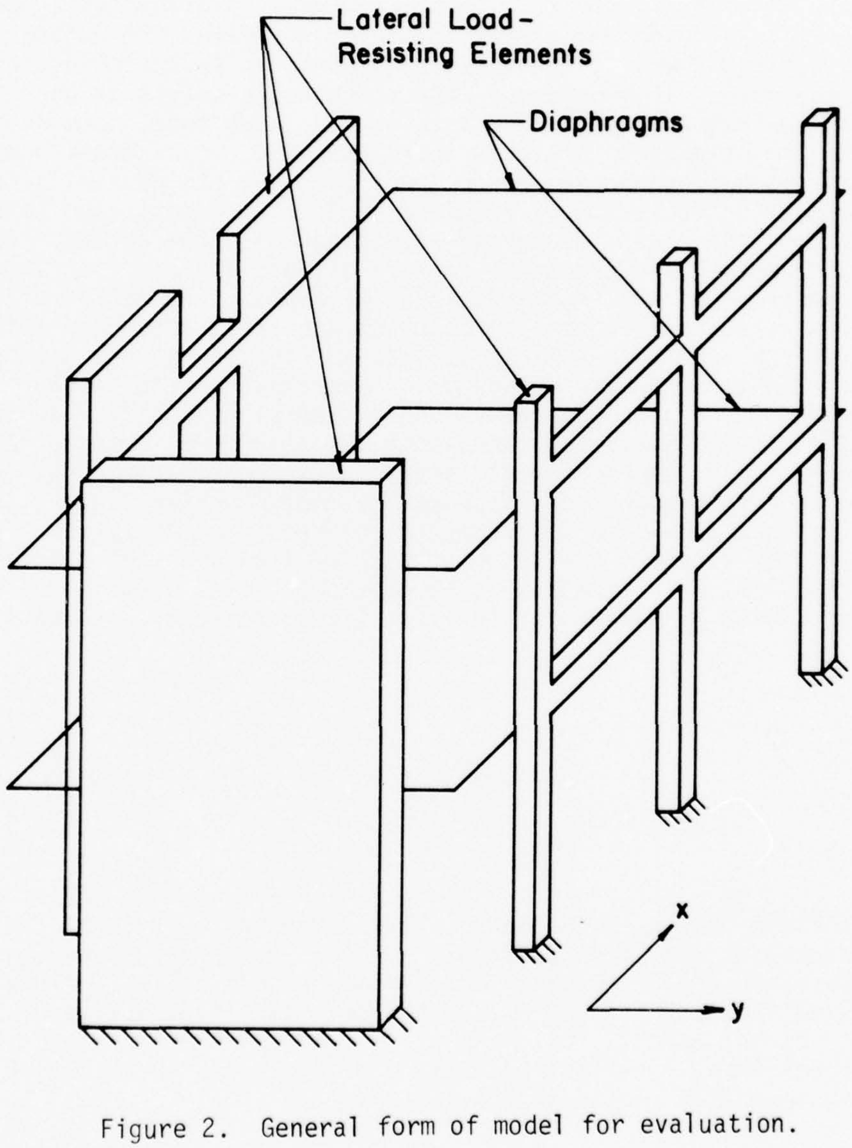

Figure 2. General form of model for evaluation.

story to story and complicated plan configuration.[7-10] Examples of poor behavior due to irregular or unusual configurations are discussed in several publications.[11-13] The following paragraphs discuss several aspects of this problem in greater detail.

Torsion

Buildings with irregular or unusual configurations will exhibit torsional effects (twisting about a vertical axis). Torsion will occur at the level of a given diaphragm when the diaphragm's center of mass does not correspond to the center of stiffness of the lateral load-resisting elements. Except for very nonuniform mass distributions, symmetrical layout such as that shown in Figure 3a will have little tendency for torsional response. An irregular arrangement however, such as that shown in Figure 3b, will invariably produce significant torsional response. Appendix E provides a description of a building's torsional response.

It must be noted that a certain amount of eccentricity between the centers of mass and stiffness is considered for all buildings, even those that are symmetrical. The calculations for stiffness and mass distribution are of insufficient accuracy to warrant an assertion of zero eccentricity. A minimum of 5 percent of the greater dimension of the building must be used for eccentricity or horizontal torsional moment.

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[7] A. K. Chopra, V. V. Bertero, and S. A. Mahin, "Response of Olive View Medical Center Main Building During San Fernando Valley Earthquake," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[8] L. G. Selna, M. D. Cho, and R. K. Ramanathan, "Evaluation of Olive View Hospital Behavior on Earthquake Resistance Design," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[9] V. V. Bertero, B. Bresler, L. G. Selna, A. K. Chopra, and A. V. Koretsky, "Design Implications of Damage Observed in Olive View Medical Center Buildings," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[10] H. S. Lew, E. V. Leyendecker, and R. D. Dikkers, *Engineering Aspects of the 1971 San Fernando Earthquake*, Building Science Series 40 (National Bureau of Standards, December 1971).

[11] S. S. Tezcan, M. Ipek, and S. Acar, "Reasons for the Earthquake Damage to the New High School Building in Burdur, Turkey," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[12] A. R. Flores, "The Luzon Earthquakes of August 2, 1968 and April 7, 1970," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[13] Architectural Institute of Japan, *Design Essentials in Earthquake Resistant Buildings* (Elsevier Publishing Company, 1970).

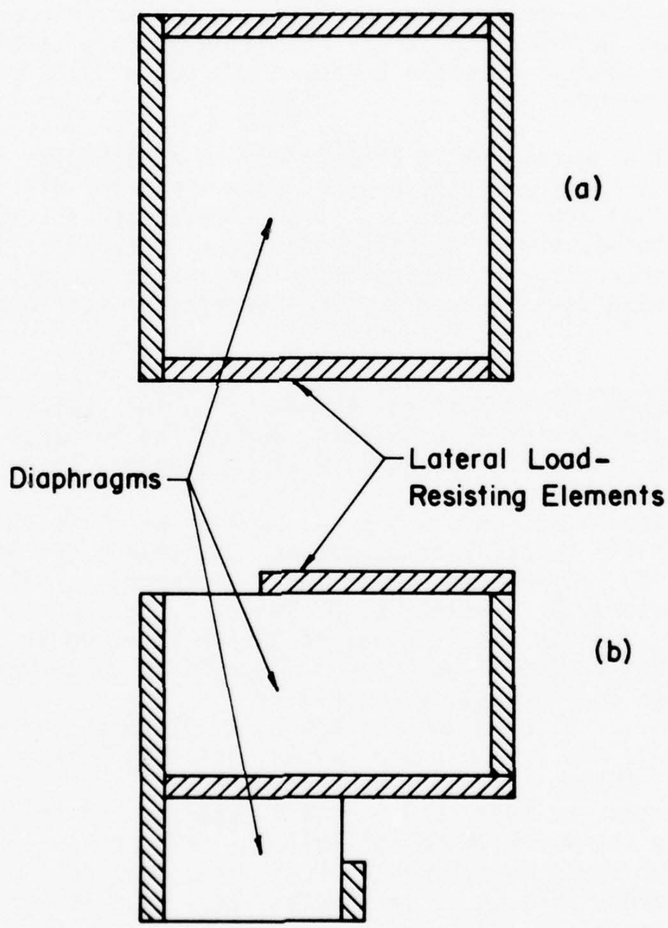

Figure 3. Comparison of symmetrical and irregular lateral load-resisting systems.

Stress Concentrations

Stress concentration problems can be induced by the geometry of irregularly shaped buildings. One such problem occurs at the reentrant corners of L- and T-shaped buildings (Figure 4). Under torsional loading, the stress at such a concave corner will be higher than that along a straight portion of wall, and much higher than that at a convex corner. The distribution of stress along the wall is not uniform, which is consistent with the theory of thin-wall, noncircular sections.[14]

A problem can also develop when a narrow corridor or connection joins two wings of a building (Figure 4). It is likely that the two wings will not deflect perfectly in phase. As a result, the connection may become a region of high stresses and deformations and therefore, a region of heavy damage.

Stress concentrations may also develop where a structure is joined to a much smaller structure, such as in a "lean-to" structure.[15]

Changes in Stiffness With Height

A structural framing system with a drastic change in stiffness with height is shown in Figure 5. The lateral load-resisting element consists of a frame in the first story and a wall system above the first story. The drastic change in stiffness occurs between the first and second stories.

The flexible first story is illustrated here because it is a common design concept which has not worked well under seismic loading. The energy input from the ground motion will be dissipated by ductile action of the flexible first story. The flexible story is forced to dissipate such large quantities of energy and undergo such major deformations that it sustains large amounts of structural damage. This may result in the loss of the facility, even though damage in other, stiffer stories is not severe. The first story may even collapse completely, as did the psychiatric unit at the Olive View Hospital

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[14] F. B. Seely, and J. O. Smith, *Advanced Mechanics of Materials*, Second Ed. (John Wiley & Sons, 1952).

[15] Architectural Institute of Japan, *Design Essentials in Earthquake Resistant Buildings* (Elsevier Publishing Company, 1970).

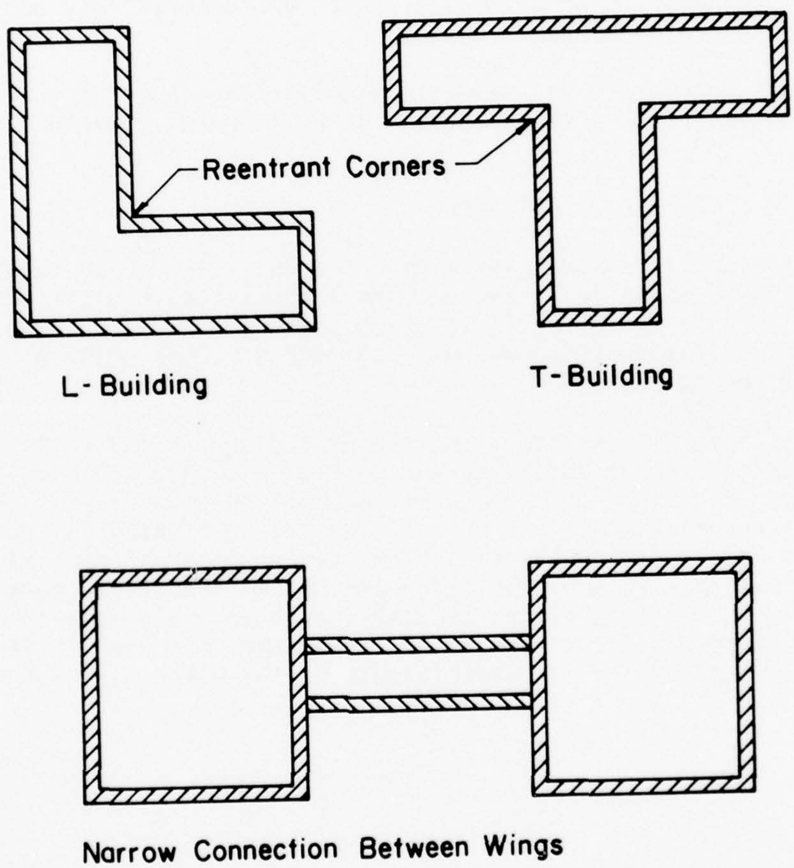

Figure 4. Building plans creating stress concentrations.

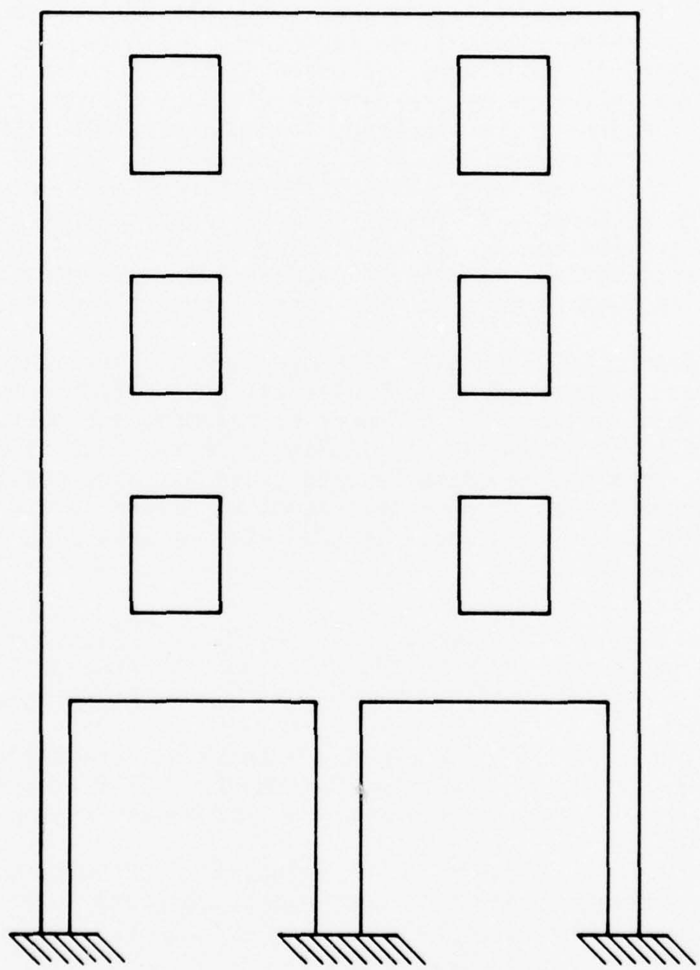

Figure 5. Example of a structural system with a change of stiffness with height.

during the 1971 San Fernando Valley Earthquake.[16-19] At any rate, the high ductilities imposed will also aggravate other problems, such as lack of confinement or inadequate shear reinforcement in columns as in the case at Olive View Hospital.

Changes in Layout With Height

Changes in layout with height can lead to several types of undesirable behavior during earthquake loading.[20]

Changes in layout will sometimes imply the termination of a wall or frame at an intermediate floor level, as in Figure 6. The horizontal plate in the figure represents a floor or diaphragm. Two supporting wall elements are monolithic with the diaphragm, and a wall at the midspan of the diaphragm from above is monolithic. The shear, V_3, and the moment, M_3, must be transferred through the diaphragm into the two lower walls. The engineer must insure that the diaphragm is capable of resisting these forces along with any other loads applied to it. If a building has many such changes in layout and terminations of lateral load-resisting elements, considerable attention must be paid to this force transfer problem.

Poor seismic behavior will also occur when a building has a large base and a tower portion (Figure 7), also referred to as a building with a "setback." The tower experiences the motion at the top of the base structure as its loading. If the base structure is quite stiff, this may be characterized by higher acceleration levels than the ground motion. The result might be higher levels of damage throughout the tower than would be predicted by analyzing the tower separately from the base.

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[16] A. K. Chopra, V. V. Bertero, and S. A. Mahin, "Response of Olive View Medical Center Main Building During San Fernando Earthquake," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).
[17] L. G. Selna, M. D. Cho, and R. K. Ramanathan, "Evaluation of Olive View Hospital Behavior on Earthquake Resistant Design," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).
[18] V. V. Bertero, B. Bresler, L. G. Selna, A. K. Chopra, and A. V. Koretsky, "Design Implications of Damage Observed in Olive View Medical Center Buildings," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).
[19] H. S. Lew, E. V. Leyendecker, and R. D. Dikkers, *Engineering Aspects of the 1971 San Fernando Earthquake*, Building Science Series 40 (National Bureau of Standards, December 1971).
[20] Architectural Institute of Japan, *Design Essentials in Earthquake Resistant Buildings* (Elsevier Publishing Company, 1970).

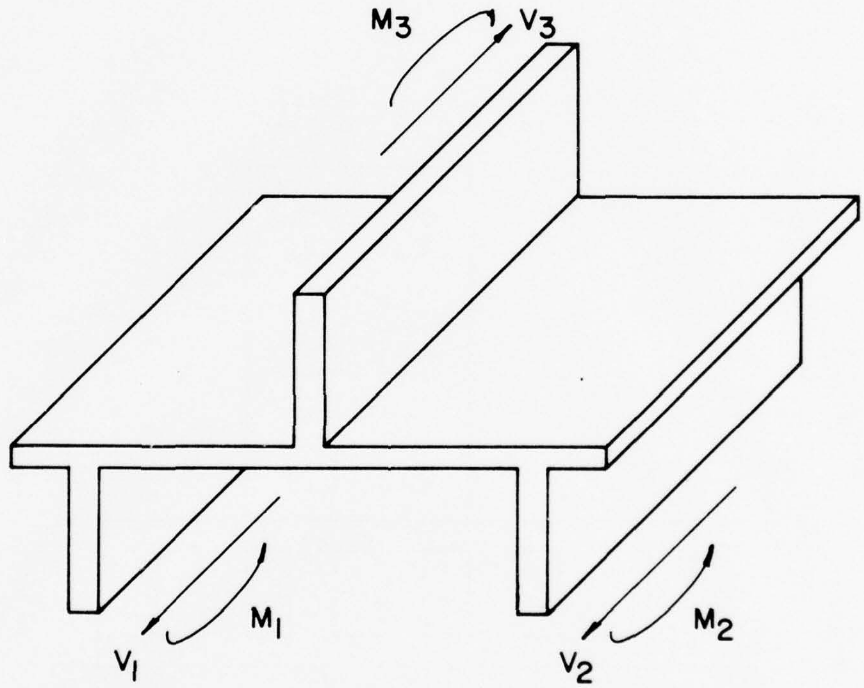

Figure 6. Force transfer at termination of lateral load-resisting element.

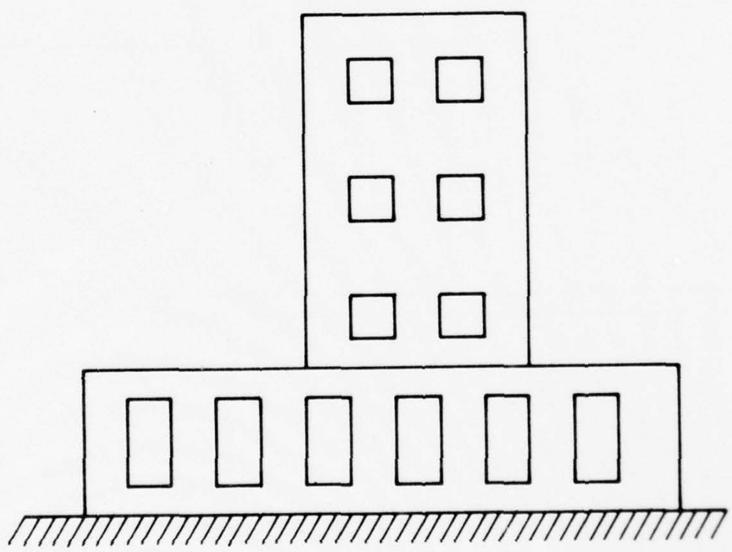

Figure 7. Building with setback.

There is even a danger of quasi-resonance occurring for any structure attached to the roof at another structure.[21] Resonance is a phenomenon that occurs when the natural frequency of the base structure is very similar to that of the tower structure. The tower structure, of course, responds to the motion of the base structure at its loading. When a structure experiences periodic motion of a frequency very close to its own natural frequency, the result is high amplitude response accompanied by large quantities of damage.

Finally, stress concentrations will occur at the tower-base interface. In order to determine with confidence the nature of force irregularities in this region, the engineer will need to exercise considerable caution in his analysis procedures. The behavior of Holy Cross Hospital during the San Fernando Valley Earthquake is an example of the results of such force irregularities.[22]

Structural System

The behavior of a structure under seismic loading depends largely on its type of lateral load-resisting system. Various materials and structural configurations differ widely in stiffness and strength, and--equally important--they differ in mode of failure and resulting capacity for ductility. Different structural systems dissipate energy in widely different ways. To an extent, the degree of redundancy inherent in the structural system is a factor in ductility. In a few cases, the yielding of one member may initiate the failure of the entire system. In other cases, other members that have not yet yielded can carry additional load, and failure does not result immediately. Some structural systems are composed of a combination of different types of lateral load-resisting elements. If one element is much stiffer initially, but less ductile than the other elements, it will attract a large portion of the lateral loads early in the earthquake and may fail. The other elements are then relied upon to carry the entire lateral loading. The engineer must consider such possibilities.

The following sections discuss the stiffness, ductility, and probable failure modes for various types of structural systems. One must, however, qualify many of the failure modes described in this chapter, since they are often the result of suppressing certain non-ductile failure modes. Needless to say, such failure modes could reduce both strength and ductility, as discussed further in Appendix B.

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[21] Architectural Institute of Japan, *Design Essentials in Earthquake Resistant Buildings* (Elsevier Publishing Company, 1970).

[22] H. S. Lew, E. V. Leyendecker, and R. D. Dikkers, *Engineering Aspects of the 1971 San Fernando Earthquake*, Building Science Series 40 (National Bureau of Standards, December 1971).

Reinforced Concrete Frames

Although frames generally have lower strengths and stiffnesses than walls or a combination system, if properly detailed they exhibit higher levels of ductility. A frame usually is highly redundant--an advantage for purposes of ductility. The desirable failure mechanism is that of flexural hinges at certain critical regions. Figure 8 shows typical failure mechanisms, and Figure 9 illustrates the ductility of this sort of mechanism. As load level increases, cracking begins at the critical regions and spreads slowly outward. Ideally, inelastic action is distributed over a large portion of the member rather than being concentrated at one section. This tends to allow higher member deformation for a given deformation at the critical section. The capacity for ductility is also dependent on the rotational capacity of the flexural hinges. Hence, it is more desirable to have the flexural hinges form in the beams (Figure 8b), rather than in the columns (Figure 8a), since the presence of axial load tends to decrease the rotational capacity of a reinforced concrete section.[23]

Finally, it should be noted that to obtain this ductile flexural failure mechanism, certain types of nonductile failure mechanisms must be suppressed. Among these are failures of members in shear, joint failures, and failure due to lack of anchorage of steel or lack of confinement of core concrete (see Appendix B).

Reinforced Concrete Walls

Lateral load-resisting systems composed of reinforced concrete walls are characterized by a small number of deep members, which results in very little redundancy. When one member fails, there may not be others to take the load. Due to the large depth of the elements, such systems generally exhibit a high level of stiffness. This may help control nonstructural damage if the walls do not fail; however, in systems with a combination of walls and frames, it may cause the walls to attract virtually the entire lateral load, causing the walls to fail and leaving only the frame to carry the load.

Figure 10 shows the failure mechanisms for a reinforced concrete wall. The figure depicts a plain prismatic structural wall with no openings or boundary elements. When a lateral load is applied at the top, failure may be characterized by a flexural hinge near the base (Figure 10a) or by a shear failure (diagonal tension) as in Figure 10b. Which mechanism occurs depends on the reinforcement and the height-to-depth ratio. In either case, the formation of the first flexure or

[23] J. A. Blume, "Design of Earthquake Resistant Poured-in-Place Concrete Structures," *Earthquake Engineering*, R. L. Wiegel, Coordinating Editor (Prentice-Hall, Inc., 1970).

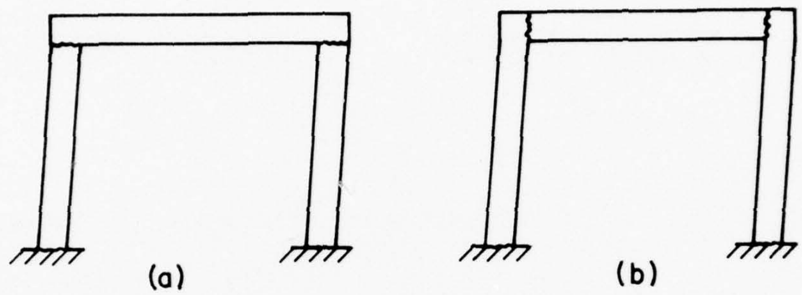


Figure 8. Failure mechanisms for reinforced concrete frames.

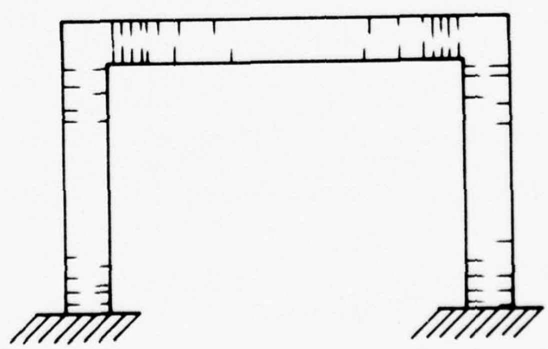
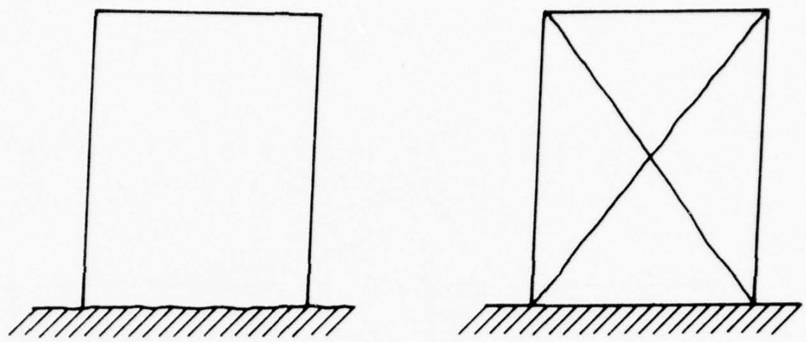


Figure 9. Crack pattern for reinforced concrete frame.



(a) (b)

Figure 10. Failure mechanisms for reinforced concrete wall.

shear crack often means the failure of the element. Ductility is somewhat dependent on the steel ratio and the confinement of core concrete; however, for a wall without boundary elements, it is quite low compared to that of a frame. It has been suggested that a reinforced concrete wall without a boundary frame must resist lateral loads without cracking.[24] If a structural wall has openings for doors or windows, it is important that there be reinforcement around the perimeters of the openings, since stress concentrations may exist around such openings, if there is a region where cracks can form.[25]

Unreinforced Masonry

Unit masonry that lacks reinforcing steel generally fails under seismic loading. Such lateral load-resisting systems are very stiff and brittle, attract high force levels, and have no reliable tensile strength. Failure may be due to either diagonal tension or flexural tension, but in either case there is no ductility. This is reflected in the requirements of the American Concrete Institute (ACI).[26] *Earthquake Engineering*[27] provides several vivid examples of the poor performance of this type of construction under seismic loading.

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[24] B. Bresler, *Behavior of Structural Elements - A Review*, Building Science Series 46 (National Bureau of Standards, February 1973).

[25] B. Bresler.

[26] *Building Code Requirements for Reinforced Concrete*, ACI Standard 318-71 (American Concrete Institute, November 1971).

[27] K. V. Steinbrugge, "Earthquake Damage and Structural Performance in the United States," *Earthquake Engineering*, R. L. Wiegel, Coordinating Editor (Prentice-Hall, Inc., 1970).

Reinforced Masonry

In this type of construction, reinforcing steel acts with the masonry to withstand loading. The reinforcement may be placed in the voids in the unit masonry and grouted, or it may be placed between two wythes of masonry and grouted (the grout provides for transfer of forces between the steel and masonry). If the reinforcement is properly arranged and grouted so that there is proper bonding between steel and grout and between grout and masonry, a system of reinforced masonry may behave similarly to a reinforced concrete wall.

In many cases, the material strength of such a system may be difficult to estimate without tests. Failure may occur in the mortar or in the masonry units, depending on their relative strengths. Rather extensive recommendations for testing reinforced masonry specimens for shear strength, compressive strength, and tensile strength have been developed.[28] Such testing naturally requires the removal of several specimens of various sizes from the structure, strength being partially a function of specimen size. Results become more reliable with an increase in specimen number, but damage to the structure also increases. Methods of nondestructive testing[29] may be difficult to apply, due to the composite nature of the construction.

The overall failure mechanisms for a reinforced masonry element would be similar to those for a reinforced concrete wall (Figure 10). It does appear to help ductility somewhat if failure is in bending rather than in shear.[30]

Reinforced Concrete Walls With Boundary Elements

Many reinforced concrete walls have their bending and shear strengths concentrated near their perimeters or boundaries. Such walls may range from a wall of prismatic shape with a concentration of reinforcement at the boundaries (Figure 11a shows a cross section), to a wall with flanges at its boundaries (Figure 11b). The latter case may be thought of as a reinforced concrete wall cast monolithically with a surrounding frame. Necessary ties are required for the vertical boundary reinforcing.

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[28] S. G. Fattal and L. E. Cattaneo, "Evaluation of Structural Properties of Masonry in Existing Buildings," *Earthquake Resistant Design Requirements for VA Hospital Facilities*, Report of the Earthquake and Wind Forces Committee (Veterans Administration, Office of Construction, March 1975).

[29] M. V. Maholtra, "In-Place Evaluation of Concrete," *ASCE Journal of the Construction Division*, Vol 101 (June 1975).

[30] R. Meli, "Behavior of Masonry Walls Under Lateral Loads," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

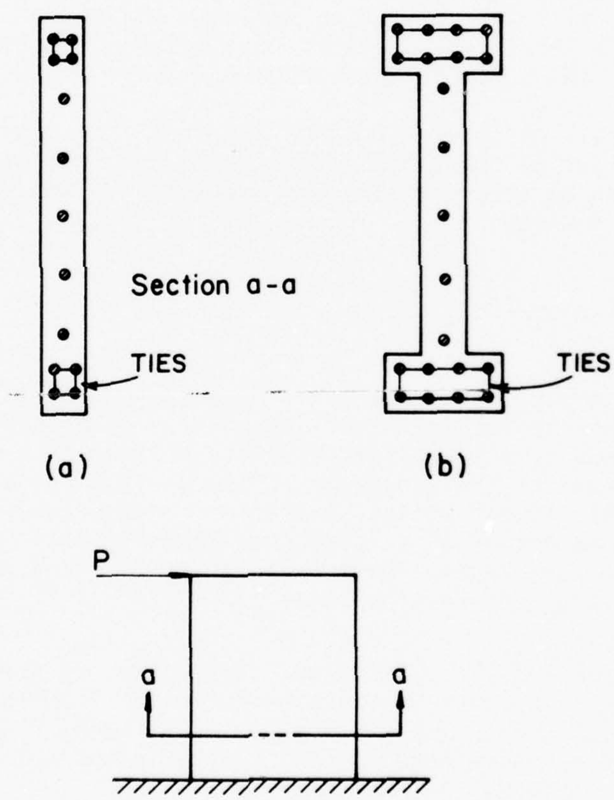

Figure 11. Structural wall with boundary elements.

The presence of a boundary element can significantly increase ductility of the system over that for a wall without boundary elements. If such an increase in ductility is to be realized, however, the boundary element must be able to act as a frame to carry the applied lateral loading as the wall interior deteriorates. The concept is that as the wall portion develops diagonal cracks, the boundary element carries additional load and experiences an array of well-distributed flexural cracks. The result is a ductile flexural failure in the frame, rather than a less ductile wall failure.[31,32] There should still be reinforcement in the interior portion, however, to control crack width and prevent complete destruction of the interior. It should also be remembered that when the system acts in flexure, large axial forces due solely to lateral loading develop in the boundary elements (Figure 12). The engineer must insure that the boundary element can carry its net axial loads, considering both vertical and lateral loading.

Infilled Frames

An infilled frame system as considered here consists of a reinforced concrete frame with unit masonry filler walls (Figure 13). The first question to be addressed for such a system concerns the degree of composite action that may be expected to occur between the frame and the infilled wall. For purposes of stiffness and strength determination, it must be decided whether the system is a true composite, or whether it is merely a frame and a separate masonry wall on the frame's interior.

Analogous to the case for a reinforced concrete wall with boundary elements, a higher frame strength relative to wall strength produces improved capacity for ductility.[33] Figure 14 shows two failure mechanisms for an infilled frame system. When diagonal cracking occurs in the masonry of a strong frame, the frame attracts additional lateral load, and the failure mechanism is that of a ductile flexural failure in the frame. If the frame is much weaker, however, the diagonal cracking of the masonry will extend through the frame. Ductility will be similar to that of a masonry wall.

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[31] R. Barda, J. M. Hanson, and W. G. Corley, "An Investigation of the Design and Repair of Low-Rise Shear Walls," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).
[32] B. Bresler, *Behavior of Structural Elements - A Review*, Building Science Series 46 (National Bureau of Standards, February 1973).
[33] R. Meli, "Behavior of Masonry Walls Under Lateral Loads," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

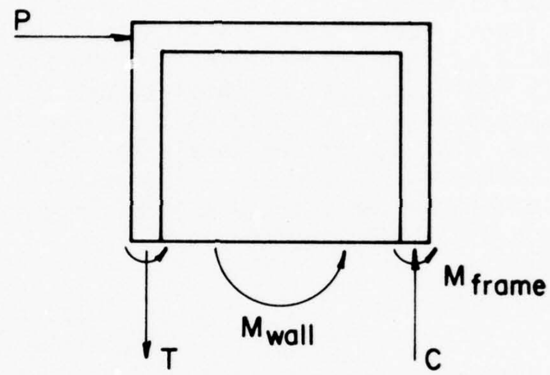

Figure 12. Induction of axial forces in frame.

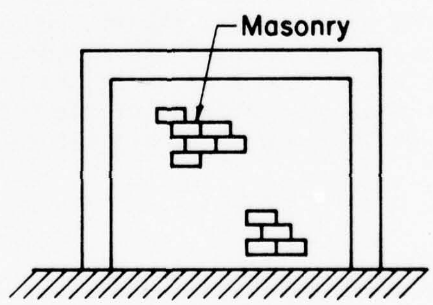

Figure 13. Infilled frame.

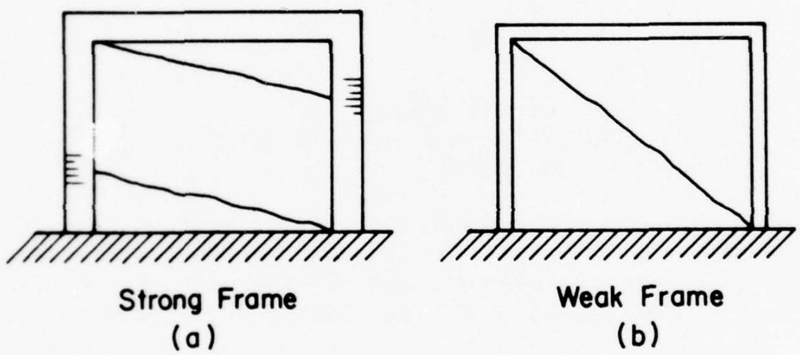

Figure 14. Effect of frame strength on failure mechanism.

Figure 15 depicts a ductile braced frame (strong frame type) failure mechanism proposed by Lefter.[34] The masonry in the middle of the interior is assumed to be lost, and only the masonry represented by the hatched triangles is assumed to remain. These are the implied limits of diagonal cracking. The intact masonry wedges act as braces for the frame; hinges form in the frame at the indicated locations. Note that this mechanism, although basically ductile due to its flexural nature, implies a reduced effective length for the columns. Hence, if one believes that the system can form this mechanism, one must be especially conscious of the shear capacity of the columns. Another secondary problem with this mechanism is the lateral stability of the masonry wedge. Figure 16, which shows the idealized loading on the wedge, may be thought of as an enlarged view of the lower masonry wedge of Figure 15. The portion of the column below plastic hinge A applies the pressure, σ, uniformly over the bearing surface between the wedge and the column. For stability of the masonry wedge, Lefter suggests

$$\frac{e}{t} \leq \sqrt{\frac{\pi^2 E_w}{96\sigma}} \qquad \text{[Eq 1]}$$

where $e = \frac{h}{4}$ (the height of the wedge)
E_w = Young's Modulus for the wedge material
t = thickness of the wedge.

Furthermore, the lower edge of the wedge must be able to carry the entire shear, V. This is the total shear at plastic hinge A. Lefter has also proposed elaborations of this mechanism for cases of several bays and for cases where the masonry infill has openings for doors or windows.

Coupled Structural Walls

Most structural walls have openings for doors or windows. If the area of the openings is relatively small compared to the wall area, it may be best to analyze the system as a plain structural wall, reducing stiffness and strength somewhat to account for the effects of the openings. Figure 17a represents such a case. As the openings become larger, however, the system is transformed into a series of smaller walls, or piers, connected by short, deep beams, or lintels

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[34] J. Lefter and J. Colville, "Reinforcing Existing Buildings to Resist Earthquake Forces," *Cento Symposium on Earthquake Engineering and Engineering Seismology* (November 1974).

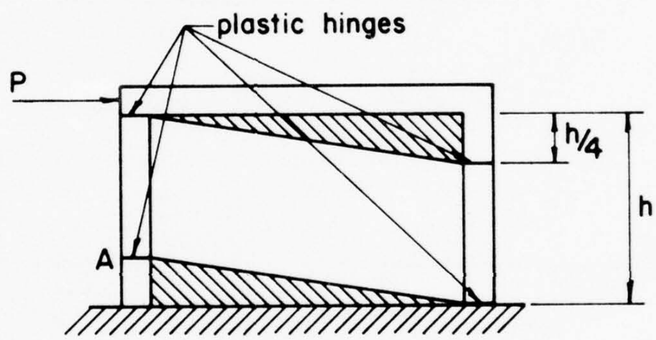

Figure 15. Braced frame failure mechanism.

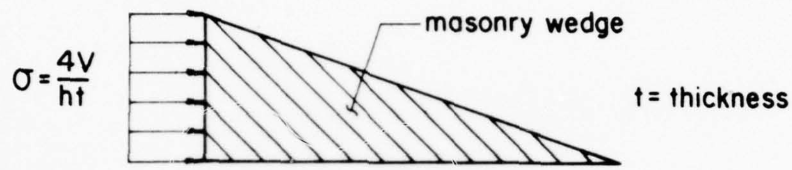

Figure 16. Loading on masonry wedge.

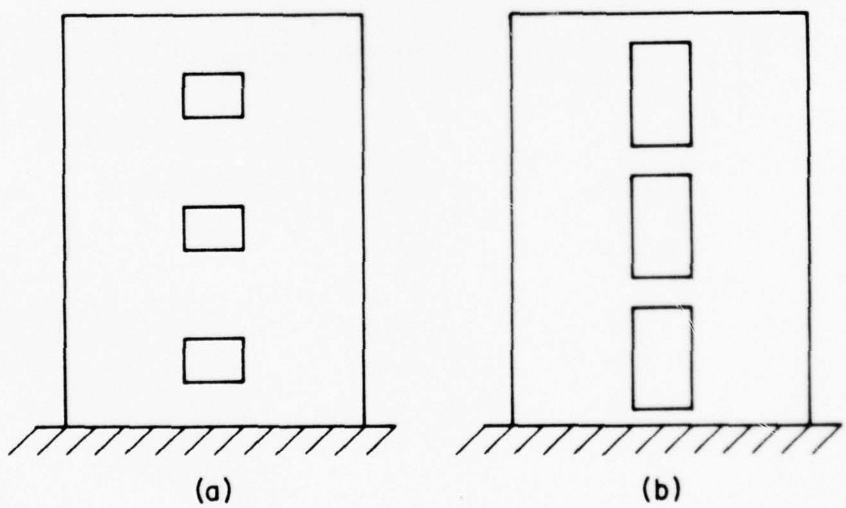

Figure 17. Structural walls with openings.

(Figure 17b). In many cases, to provide the boundary steel around the openings, the lintels must be reinforced like beams, so this system becomes one of several walls connected by deep beams.

There are several possible failure mechanisms for coupled wall systems. Figures 18a and 18b represent shear failures in the connecting lintels and in the piers, respectively. Figure 18c represents a failure in axial tension at the base of the piers, which may occur in cases where the flexural strength of the connecting lintels is very high and the lintels are designed to handle the accompanying high shear. These three mechanisms tend to be brittle. The capacity for ductility may be similar to that of an isolated structural wall. The failure mechanism in Figure 18d is characterized by flexural failures at the ends of the connecting beams and at the bases of the piers. Since this mechanism is flexural, it tends to be somewhat more ductile than the others.

Other Wall-Frame Interaction Systems

Two other examples of wall-frame interaction systems are shown in Figure 19. Figure 19a is a wall portion of a lateral load-resisting system attached directly (rather than indirectly through a diaphragm) to a frame, which is along the same axis as the wall. Figure 19b shows a wall and a frame that are side by side and connected to a diaphragm.

Since there are many varieties of interaction systems, detailed problems of their behavior vary greatly, and the engineer must exercise discretion when analyzing them. One general problem common to many of these systems is the fundamental incompatibility in stiffness and ductility between the wall and the frame. It is very possible that since the wall is much stiffer than the frame, it will attract virtually the entire lateral load early in the seismic loading and fail at low ductility levels. The frame might then be forced to carry virtually the entire lateral load and either fail or be required to attain such large deflections that nonstructural damage is enormous. The engineer must try to ascertain what portion of the load will initially be distributed to various portions of the system and make an analysis on this basis. Additional analyses that assume partial or total destruction of the wall may be warranted.

Braced Frame

A one-story, one-bay braced frame is shown in Figure 20. The cross-rods provide the reinforced concrete frame with additional resistance to overall shear deformation. The structural action of the bracing shown in Figure 21 can be compared to that of a low wall acting in shear. The bracing rods act similarly to the region of diagonal tension and compression in the wall. Hence, in some cases, the stiffness of the braced frame may be similar to that of a wall.

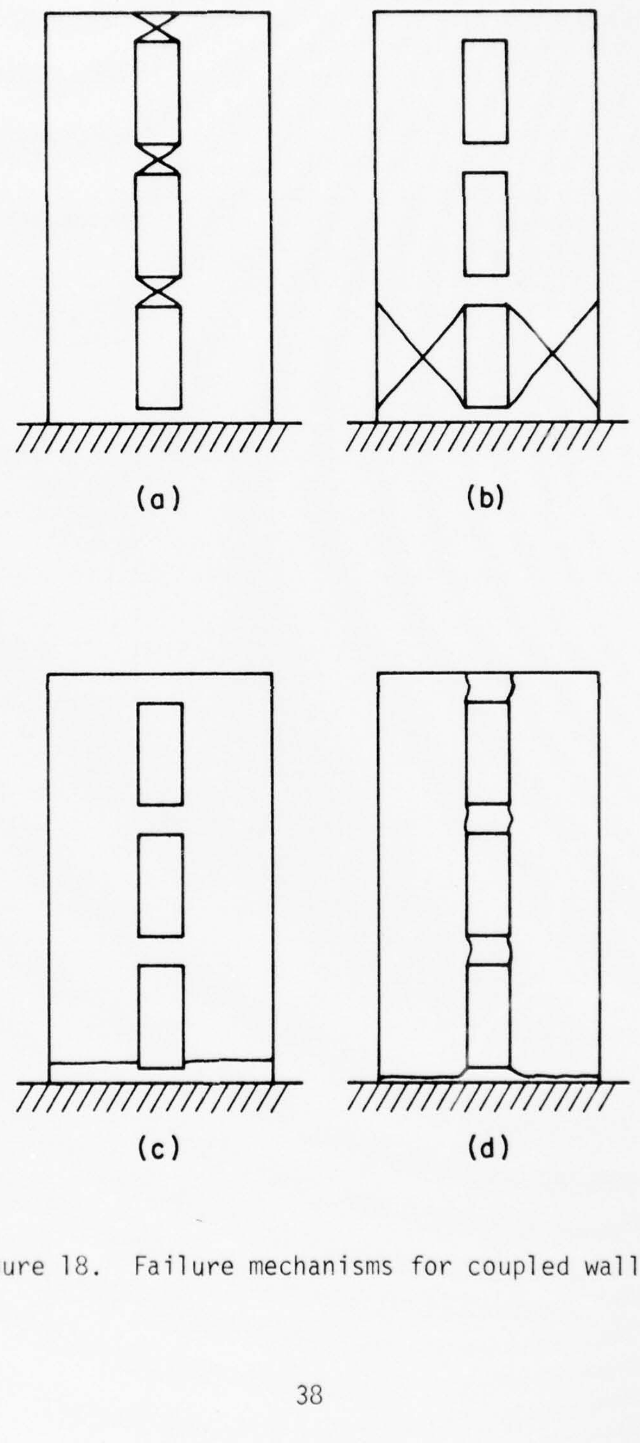

Figure 18. Failure mechanisms for coupled walls.

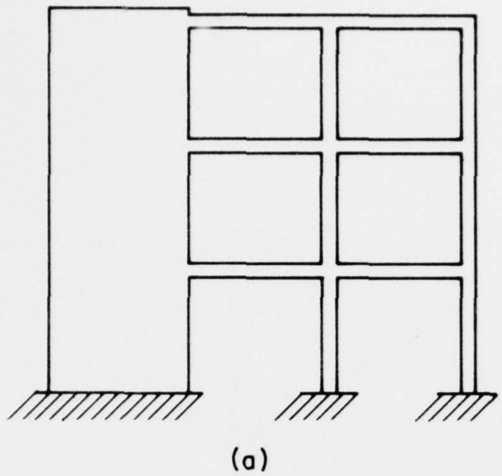

Figure 19. Other examples of wall-frame interaction.

However, the engineer must consider the mode of system failure. In some cases, the bracing may fail first, leaving a much-reduced lateral load resistance, even though the wall may be ductile. In other cases, the frame may form its ductile flexural failure first.

A problem that should be considered for these systems is stress concentrations at the ends of the bracing.[35] The engineer should be sure that stress levels in this region are conservative to insure that the bracing will not fail.

Diaphragms

Diaphragms must transfer the inertial forces to the lateral load-resisting elements or must transfer forces from one lateral load-resisting element to another. The strength and stiffness of the diaphragm are important parts of the structural evaluation. Unfortunately, the information on these topics appears to be quite sparse.

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[35] Architectural Institute of Japan, *Design Essentials in Earthquake Resistant Buildings* (Elsevier Publishing Company, 1970).

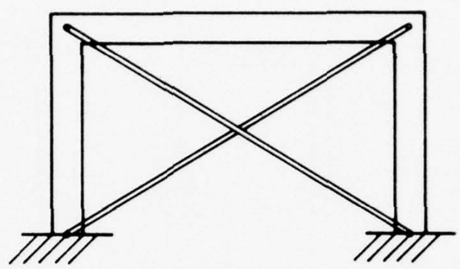

Figure 20. Braced frame.

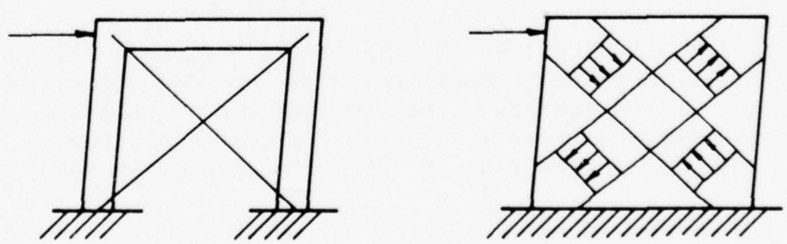

Figure 21. Comparison of braced frame and wall.

Stiffness

The stiffness of the diaphragm will affect the distribution of forces among the lateral load-resisting elements (Figure 22).

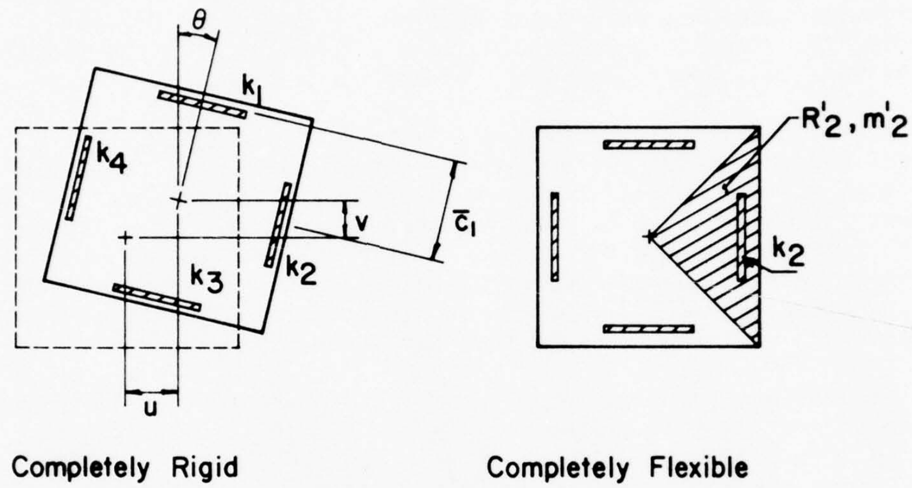

Figure 22. Comparison of force distributions with rigid and flexible diaphragms.

Consider, first, the completely rigid diaphragm. If response in the u-direction is considered, the deflection of the ℓth lateral load-resisting element is given in Figure 22 by

$$u_\ell = u + \bar{c}_\ell \theta \qquad \text{[Eq 2]}$$

The shear force transmitted by the lateral load-resisting element would be given by

$$V_\ell = k_\ell u_\ell = k_\ell u + k_\ell \bar{c}_\ell \theta \qquad \text{[Eq 3]}$$

Hence, for an infinitely rigid diaphragm, each lateral load-resisting element carries load in proportion to its stiffness. The diaphragm effects a force transfer from one region of the building plan to another to maintain this relation.

For a very flexible diaphragm, however, there can be little transfer of forces from one region to another. Each lateral load-resisting element must transmit the inertial loads generated by adjacent regions in the building plan. Lateral loads are no longer distributed according to element stiffness. In Figure 22, for example, the element corresponding to $\ell = 2$ would carry the inertia of the mass, m_2', associated with its tributary area, R_2'.

Having a stiff diaphragm system is a distinct advantage. As a lateral load-resisting element yields (is damaged), its stiffness decreases. Since a stiff diaphragm distributes forces according to element stiffness, it will transfer load to less damaged elements, so the damage is more distributed. A stiff diaphragm can also partially compensate for concentrations of mass. One publication has suggested that all diaphragms should be carefully inspected for cracks--even shrinkage cracks--that may reduce the stiffness.[36]

At present, the best readily available approach to determining the stiffness level of a diaphragm is the flexibility factor, F, used in TM 5-809-10 in conjunction with Table 2-4-1 of that publication. Wood diaphragm systems are almost always very flexible. This flexibility is very dependent on the quality of nailing and connections; however, these systems should be assumed to distribute loads by tributary area. If a diaphragm is composed of precast segments, the quality of the connections will exert a major effect on the diaphragm's overall stiffness. The engineer should try to ascertain the extent of this effect.

Strength

The best available means of calculating a diaphragm's strength appears to be that outlined in TM 5-809-10--the use of the variable q_D. Figure 23 is a guide for analyzing a diaphragm's strength. The diaphragm should be analyzed as a deep beam, considering forces V_1, V_2, and V_3 to be the maximum lateral load capacity of the respective elements. The forces M', T', and V' at various sections in the diaphragm are then computed by statics. Additional consideration should be given to a diaphragm that has deep-edge elements (lower portion of Figure 23). The moment, M', may cause large axial forces, C" and T", to occur in the edge elements. The engineer should be sure that the edge elements can carry these forces.

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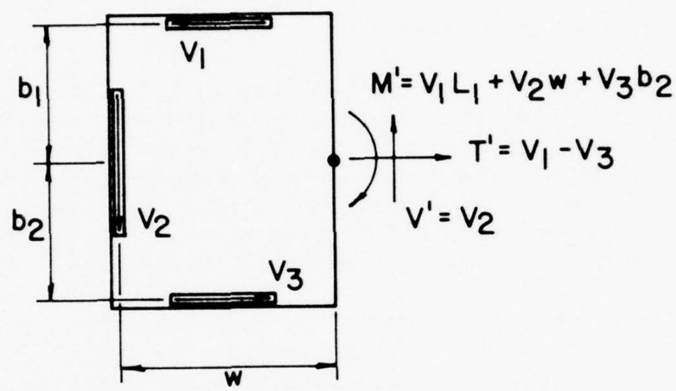
[36] Architectural Institute of Japan, *Design Essentials in Earthquake Resistant Buildings* (Elsevier Publishing Company, 1970).



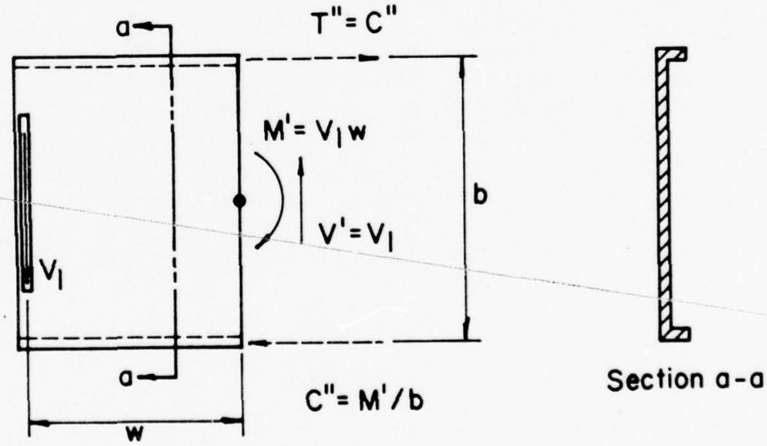


Figure 23. Forces in a diaphragm.

3 CASES FOR EVALUATION AND COURSE AND EXTENT OF EVALUATION

For some structures, it may be desirable to perform analyses for several cases--several distributions of stiffness or strength throughout the structure. Analyses would be performed for the complete intact structural system with consideration for certain attached members or elements contributing to the strength or stiffness. Additional analyses would be performed for the structural system with certain elements reduced in strength or stiffness, simulating structural damage to those elements.

The above considerations may be especially important for structures which exhibit radical variations in stiffness among their lateral load-resisting elements. The stiff element may fail or be severely damaged before the more flexible element will carry significant load. An example is a wall and a frame connected by a diaphragm, as in Figure 19b. If the two elements experience similar deflections early in the loading sequence, the wall will attract the majority of the total lateral load. Being considerably less ductile, the wall may fail completely, while the force resisted by the frame is small. However, with the wall destroyed, the frame is compelled to carry the entire lateral load. Such problems may also occur with infilled frames (Figure 14) and frames directly attached to walls (Figure 19a).

It is important to ascertain how the structure might perform in a partially damaged condition. This can be learned by evaluating several strength or stiffness cases. Having decided what stiffness or strength cases to evaluate, it must be decided, for each case, (1) how detailed an evaluation to perform; (2) to what extent structural details will be investigated; (3) whether to use the approximations for simple structures or perform a complete modal analysis (Appendix C); and (4) whether to decide that the structure is extremely simple and noncritical and the results of the general observations and investigations are extremely conclusive.

4 ANALYSIS FOR LINEARLY ELASTIC RESPONSE

This chapter provides the procedure for using a design response spectrum and modal analysis techniques to compute a base shear consistent with linearly elastic response.

Preparation for Analysis

1. Idealize structure. Consider the mass distribution, the manner in which the structure is to be apportioned into separate lateral load-resisting elements, the manner in which coupling of walls and frames is to be considered, what nonstructural parts will act structurally, and the degree to which torsional response will be considered.

2. Stiffnesses. Compute section or element stiffnesses as needed for input to structural analysis computer program.

Simplified Analysis

1. Period. Compute the period, T_1, from the formulas below:

a. Reinforced concrete frames: $T_1 = 0.1\ n$ (from TM 5-809-10).

b. Buildings with 50 percent or more of base shear carried by walls: $T_1 = 0.05\ n$.

c. Buildings with masonry walls: $T_1 = 0.05\ n$.

Note: If $T_1 > 0.35$ sec, do not use this simplified analysis method. Compute the frequency of the mode, f_1, from $f_1 = \frac{1}{T_1}$.

2. Mode shape. Choose a linear mode shape, ϕ_{ij}. In essence, choose the values $\phi_{11}, \phi_{12}, \ldots, \phi_{1n}$ such that they are proportioned to their height above the base of the structure.

3. Participation factor. Compute the participation factor, c_1, for the mode from

$$c_1 = \frac{m_1\phi_{11} + m_2\phi_{12} + \ldots + m_n\phi_{1n}}{m_1\phi_{11}^2 + m_2\phi_{12}^2 + \ldots + m_n\phi_{1n}^2} = \frac{\sum_{j=1}^{n} m_j\phi_{1j}}{\sum_{j=1}^{n} m_j\phi_{1j}^2} \qquad \text{[Eq 4]}$$

Full Modal Analysis

1. Computer program. Obtain the results in items 2 through 4 below by using an appropriate modal analysis program, e.g., STRUDL from the ICES Package (Massachusetts Institute of Technology) or SAP (University of California, Berkeley).

2. Frequencies. Obtain f_i for $i = 1, 2, \ldots, m$.

3. Mode shapes. Obtain ϕ_{ij} for $i = 1, 2, \ldots, m$ and $j = 1, 2, \ldots, n$.

4. Participation factors. Obtain c_i for $i = 1, 2, \ldots,$

Response Spectrum

1. Obtain the design response spectrum previously developed for the site in question.

2. Note the damping factor, β_s, for which the design response spectrum has been developed.

3. Obtain the spectral acceleration, $\ddot{s}_i$, for each frequency f_i, where $i = 1, 2, \ldots, m$.

4. Compute the relative single-degree-of-freedom (SDF) responses from

$$\ddot{\xi}_i = \ddot{s}_i - \ddot{x}_b \text{ for } i = 1, 2, \ldots, m \qquad \text{[Eq 5]}$$

where $\ddot{\xi}_i$ = acceleration realtive to the ground for the single-degree-of-freedom system corresponding to the i^{th} mode
$\ddot{x}_b$ = the base acceleration.

Linearly Elastic Inertial Floor Loads

1. Modal accelerations. Compute the maximum acceleration relative to the ground for the i^{th} mode at the level of the j^{th} diagram, $\ddot{w}_{ij}$, from

$$\ddot{w}_{ij} = c_i \phi_{ij} \ddot{\xi}_i \text{ for } i = 1, 2, \ldots, m \text{ and } j = 1, 2, \ldots, n. \qquad \text{[Eq 6]}$$

2. Combination of modes. Compute the acceleration at each story relative to the base, $\ddot{u}_j$, from

a. Two modes:

$$\ddot{u}_j = c_1 \phi_{1j} \ddot{\xi}_1 + c_2 \phi_{2j} \ddot{\xi}_2 \text{ for } j = 1, 2, \ldots, n. \qquad \text{[Eq 7]}$$

b. Three or more modes:

$$\ddot{u}_j = \sqrt{\sum_{i=1}^{m} \ddot{w}_{ij}^2} \quad \text{for } j = 1, 2, \ldots, n. \qquad \text{[Eq 8]}$$

3. Linearly elastic accelerations. Compute the absolute acceleration at each story, $\ddot{x}_j$, from

$$\ddot{x}_j = \ddot{u}_j + \ddot{x}_b, \text{ for } j = 1, 2, \ldots, n. \qquad \text{[Eq 9]}$$

4. Linearly elastic story loads. Compute the applied inertial force from linearly elastic assumptions at each level, P_j, from

$$P_j = m_j \ddot{x}_j \text{ for } j = 1, 2, \ldots, n. \qquad \text{[Eq 10]}$$

Distribution of Loads to Individual Lateral Load-Resisting Elements

1. Diaphragm stiffness. Estimate level of stiffness of diaphragms.

2. Method of load distribution. Decide whether loads will be distributed to the lateral load-resisting elements by relative stiffness or by tributary mass.

3. Distribution by stiffness.

a. Eccentricity. Compute centers of mass and stiffness for diaphragms at each level (levels 1 through n). Eccentricity at each level is the distance between these two centers.

b. Assume story load to be applied through center of mass.

c. Apply force through center of stiffness and compute necessary moment about center of stiffness to obtain equivalence to load item b.

d. Compute load applied to each lateral load-resisting element, distributing in proportion to stiffness and distance from center of stiffness.

4. Distribution by tributary mass.

a. Divide each diaphragm into portions applicable to various lateral load-resisting elements.

b. Compute the weight in each portion outlined in item a.

c. Distribute the applied story load to the lateral load-resisting elements in proportion to the weights computed in item b above.

Internal Forces in Lateral Load-Resisting Elements

1. Consider each lateral load-resisting element to be subjected to statically applied lateral loads equal to the loads computed in the previous section.

2. For each element, use a static structural analysis computer program to compute the base shear, $V_{e\ell}$.

3. Compute the base shear, V_e, for the entire structure.

5 STRUCTURE CAPACITY

The following are the steps required to determine the base shear corresponding to a collapse mechanism:

1. Compute section strengths in shear and moment from the ACI Code, Strength Design. Apply appropriate strength reduction factors.

2. Determine the collapse mechanism for each lateral load-resisting element, ℓ, for $\ell = 1, 2, \ldots, z$.

3. For each element, ℓ, compute base shear, $V_{y\ell}$.

4. Compute the collapse base shear, V_y, for the entire structure.

6 STRUCTURAL DETAILS

This chapter describes the considerations which must be given to structural details such as shear in flexural members, forces in joints, and reinforcement characteristics.

Shear in Flexural Members

The flexural member to be evaluated should be loaded at its ends, as shown in Figure 24. The member of length, L, is loaded in reverse curvature by the end moments corresponding to the ultimate strength of the section, M_u. The applied shear for evaluation, V_u, is that accompanying the ultimate moment. Hence,

$$V_u = 2M_u/L \qquad \text{[Eq 11]}$$

This satisfies the concept that the member develops flexural hinges at its ends without failing in shear. It is apparent that any error in estimating section strength that causes the engineer to use an M_u that is too low may lead to underestimating the likelihood of shear failure. Since the strength of the longitudinal reinforcement has a significant effect on M_u, it is recommended that the steel strength used for the calculation of M_u be somewhat above that estimated. Similarly, the ultimate strength of the reinforcement should be used, rather than the yield strength. The shear stress in the concrete may be computed from the common truss analogy and a 45° inclined crack. This is shown in Eq 11-13 of the ACI Code. Eq 12 is an algebraic rearrangement of that equation:

$$A_v = \frac{(v_u - v_c)b_w s_v}{f_y} \qquad \text{[Eq 12]}$$

where A_v = area of shear reinforcement within a distance, s_v along a member
s_v = shear reinforcement spacing center-to-center
f_y = yield point of *shear* reinforcement
b_w = width of member web
v_u = total shear stress on section (computed from V_u)
v_c = shear stress in concrete.

Solving for v_c,

$$v_c = v_u - \frac{A_v f_y}{b_w s_v} \qquad \text{[Eq 13]}$$

Based on the computed value of v_c, the engineer must then judge the likelihood of strength degradation due to shear failure. The engineer should remember that strength degradation may be more severe if the

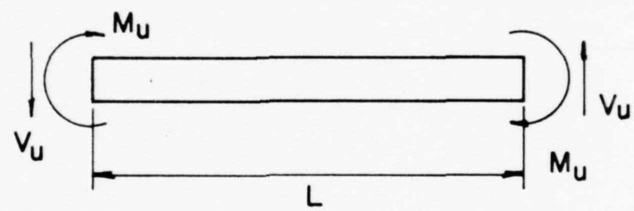


Figure 24. Loading for evaluation of shear strength of flexural member.

member is subjected to axial load, i.e., if it is a column. He/she should also be aware of the enhanced vulnerability of columns of very short effective length.

Beam-Column Joints

The engineer should first sketch a free-body diagram of the joint with applied moments corresponding to the ultimate section strengths of the members framing into the joint (Figure 25). Applied shears should correspond to these applied moments, as in the evaluation of member shear strength. The ultimate strength of the reinforcement should be used, considering possible overstrength. The usual truss analogy with a 45° tension crack should be used to evaluate the shear strength. The transverse reinforcement is assumed to be functioning at its yield stress. The engineer then computes the necessary shear stress in concrete. If deterioration is to be avoided, this stress should be very close to zero. Next, compressive stresses in the concrete should be estimated. The bond stress in the steel should be computed or the anchorage length may be compared directly with ACI requirements (Chapter 12). Again, possible overstrength of reinforcement should be considered. Finally, the engineer should note the spacing and type of confining steel and whether additional members frame into the joint laterally. Based on the above calculations, the engineer can judge whether joint failure is likely to reduce structure ductility.

Slab-Column Joints

Joints between columns and flat reinforced concrete slabs may be subject to failure under combined shear and moment. An extreme case in which the column actually punched through the slab is described in *Engineering Aspects of the 1971 San Fernando Earthquake*.[37] In some evaluations, the engineer may check the shear in the slab for such connections.

The object of evaluating slab-column joints is to estimate the shear stress in the slab in the vicinity of the column when the failure mechanism (characterized by flexural hinging) is formed. The approach suggested is that given in the ACI code and commentary, Section 11.13.[38] The total moment transferred from slab to column

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[37] H. S. Lew, E. V. Leyendecker, and R. D. Dikkers, *Engineering Aspects of the 1971 San Fernando Earthquake*, Building Science Series 40 (National Bureau of Standards, December 1971).

[38] *Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-71)* (American Concrete Institute, 1971).

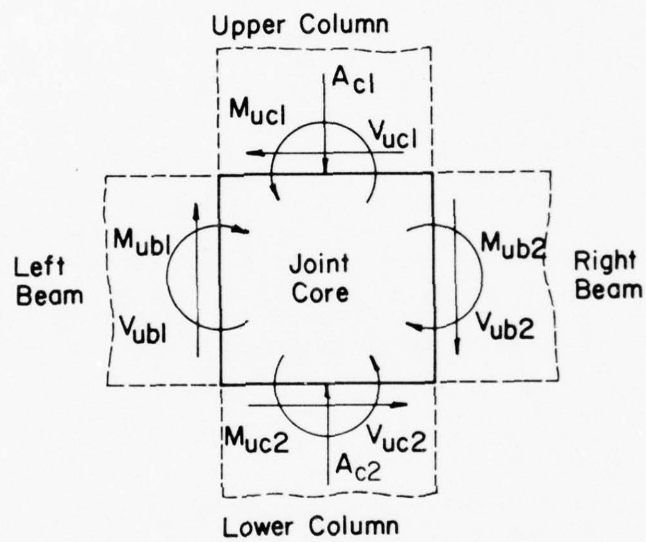


Figure 25. Loading for evaluation of beam-column joint.

will be that developed by the failure mechanism of the system, i.e., either the ultimate section strength of the column or the ultimate section strength of the slab, considered as an equivalent frame. Note that only a portion of this moment is used in the calculation. The moment may be transferred either through flexure or through eccentricity of the shearing force about the centroid of the critical section defined in Section 11.10.2 of the ACI code. Section 11.13.2 of the same publication provides a formula for the fraction of this moment transferred by eccentricity of shear. It is not apparent that this formula is strictly applicable for the case where a failure mechanism has formed, but it might be used for lack of better information. The maximum shear stress is estimated as in the ACI commentary. Based on this result, the engineer can judge the joint's adequacy.

Miscellaneous Steel Details

The confinement provided to concrete in columns should be evaluated using provisions of Appendix A of the ACI code and considering the type of confining devices present and their probable spacing.

Details such as splices in high moment regions, reinforcement in regions of stress concentration, reinforcement around openings in walls, etc., should be considered.

Anchorage lengths of reinforcement should be checked. The reinforcement should be considered to be subjected to its ultimate stress.

7 CONSIDERATION OF ENERGY DISSIPATION

This chapter provides the steps necessary to determine the required level of energy dissipation, damping and ductility and the procedure to follow in deciding whether the structure is capable of providing those levels.

Required Energy Dissipation

1. Energy dissipation factor. For each lateral load-resisting element, ℓ, where $\ell = 1, 2, \ldots, z$, compute the energy dissipation factor α_ℓ with respect to base shear from

$$\alpha_\ell = \frac{V_{e\ell}}{V_{y\ell}} \qquad \text{[Eq 14]}$$

Alternately, compute the energy dissipation factor for the entire structure from

$$\alpha = \frac{V_e}{V_y} \qquad \text{[Eq 15]}$$

2. Unusual structures. If it is thought that the structure or one of its lateral load-resisting elements may not qualify as an "energy-conserving system" as described in Appendix C (i.e., if the natural frequency is less than 0.3 hertz or greater than 20 hertz, then the required levels of ductility and damping should be determined from Tables C1 through C3 of Appendix C. Otherwise, proceed to step 3.

3. Damping and ductility. Use Figure 26 for the value of β_s for which the design response spectrum used in the analysis has been computed. Choose the interaction line for the proper value of α or interpolate between two lines. The interaction line represents the various combinations of damping and ductility that can produce the required energy dissipation.

Energy Dissipation Capacity

1. Consider the type of structural system.

2. Consider the type of yield mechanism formed.

3. Consider the overall condition or state of repair of the structure.

4. Consider the complexity and general nature of structure configuration.

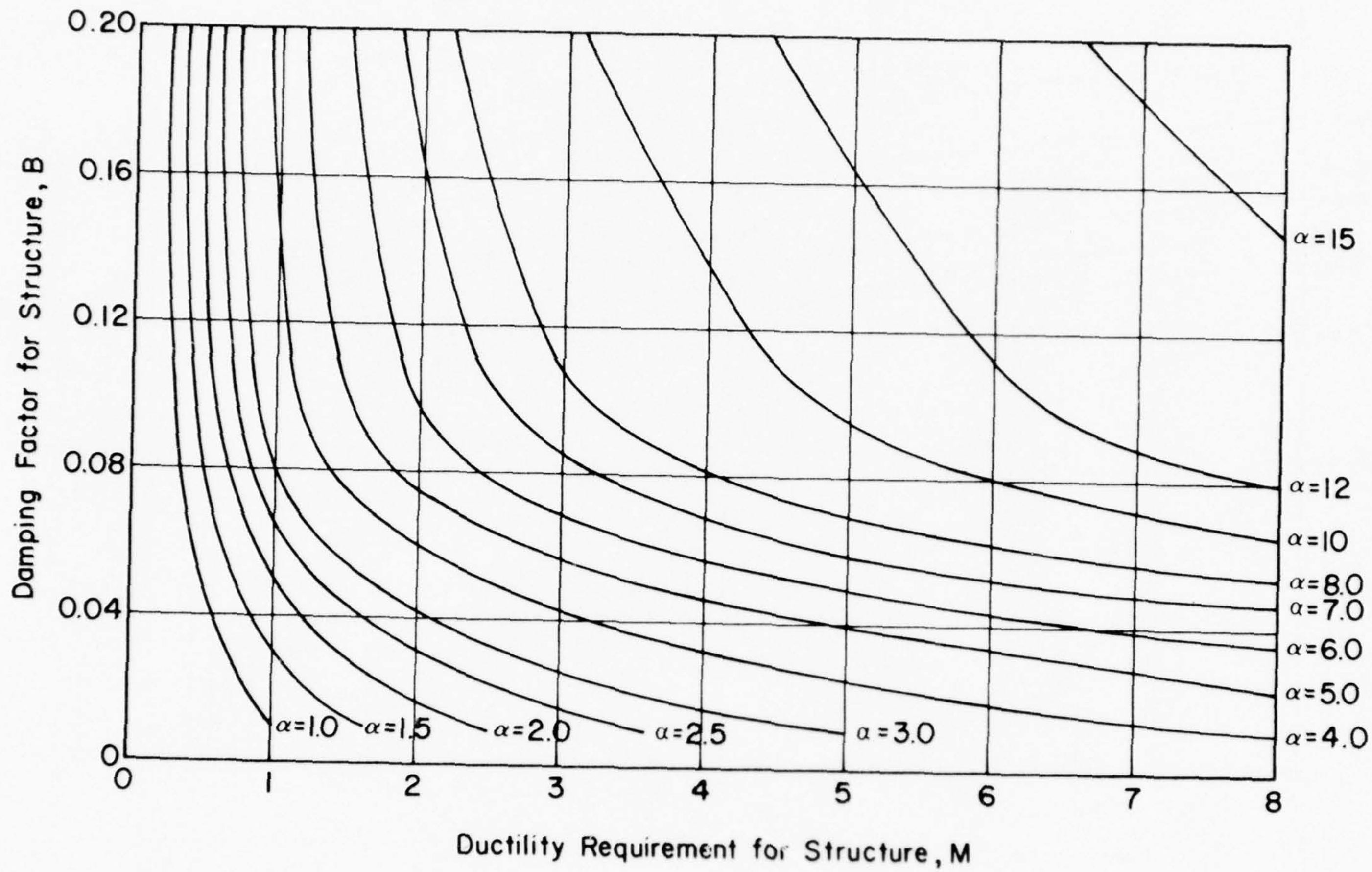
5. Consider the construction of the diaphragms.

6. Consider the structural details.

7. Make a subjective decision concerning whether the structure is capable of providing the damping and ductility levels determined in the previous section.

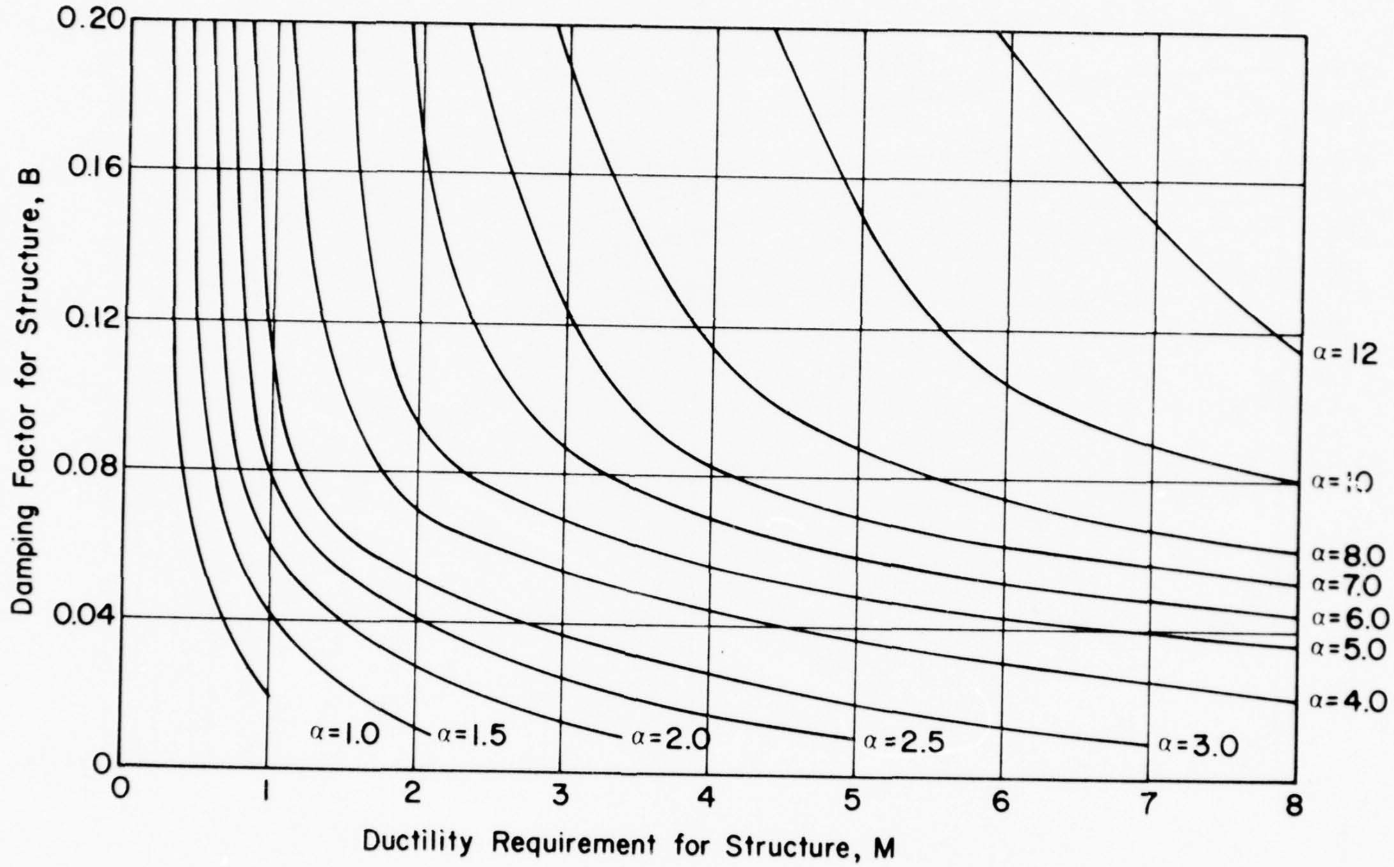
8 NONSTRUCTURAL CONSIDERATIONS

This chapter provides the procedure for determining the effects of various nonstructural characteristics.



a. $\beta_s = 0.01$

Figure 26. Interaction of damping and ductility.



b. $\beta_s = 0.02$

Figure 26 (Cont'd).

Story Drifts

1. Elastic story drifts. Compute story drifts based on the linearly elastic loads computed previously. Obtain Δ_{ej} for j = 1, 2,..., n.

2. Effect of damping. Use Table C4 and obtain γ_d for the appropriate level of damping.

3. Effect of ductility: Use Table C1 and obtain η for the appropriate ductility factor.

4. Story drift. Compute story drift from

$$\Delta_j = \frac{\eta\Delta_{ej}}{\gamma_d} \quad \text{for } j = 1, 2, \ldots, n. \qquad \text{[Eq 16]}$$

Effects of the Computed Story Drifts on Nonstructural Fixtures

Evaluate such fixtures as:

1. Glass
2. Veneers
3. Partition walls
4. Heating, ventilation, and air conditioning ducts
5. Electrical cables
6. Plumbing
7. Ceiling assemblies.

Inertial Forces in Nonstructural Fixtures

1. Compute weight of fixture.

2. Compute force, F_p, in fixture from the requirements of the Structural Engineers Association of California (SEAOC).[39]

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[39] *Recommended Lateral Force Requirements and Commentary* (Seismology Committee, Structural Engineers Association of California, 1974).

3. Consider the effect of F_p on fixtures such as:

a. Connections of facades or veneers to main structure

b. Connections of stone, tile, or various architectural ornamentation, both indoors and outdoors

c. Parapets

d. Light fixtures

e. Ceiling systems (especially hung ceilings).

9 DECISION

The information contained in Chapters 2, 4, 5, 6, 7, and 8 should now be considered together. The required ductility and damping combinations determined in Chapter 7 should be considered with respect to structural materials and type of structural system. The overall building configuration, failure mechanism, and probable adequacy of structural details should also be related to the computed damping and ductility requirements. Finally, the results of the nonstructural investigation should be input. From these data, a judgment can be made concerning the adequacy of the structure for seismic loading.

APPENDIX A:

SIGNIFICANCE OF ENERGY DISSIPATION

The concepts of inelastic structural response and energy dissipation form an integral part of the evaluation procedure presented in this report. This appendix will provide a better understanding of the significance of these concepts in seismic resistance and a greater insight into the step-by-step evaluation procedure.

Philosophy of Earthquake Resistance

The philosophy behind design codes such as the SEAOC Code has been that the structure should respond elastically to very mild earthquakes; may experience some nonstructural damage, but no structural damage, in moderately strong earthquakes; and may experience some structural damage, without collapse, in a major earthquake. It is this major earthquake that is used as the basis for both design and evaluation. The engineer wishes to insure the structure's survivability under this maximum loading event. In some cases, the structural resistance may even be evaluated for the maximum credible earthquake (the strongest earthquake that geologists estimate could ever be expected to occur at the site) as opposed to one, for example, with a 50-year return period.

The structural response considered in the evaluation process will involve yielding and considerable dissipation of energy. The traditional assumptions of linear behavior under static monotonic loading are not well suited to evaluation for seismic resistance, so the engineer must consider the inelastic action in some manner. Hence, it is not sufficient for the engineer to think only in terms of capacity for load. He/she must think in terms of a combination of load capacity and inelastic deformation.

Response of a Simple Idealized System

A single mass attached to a single column that is fixed at its base will illustrate the concept of energy dissipation. The entire building may be thought of as an assemblage of such elements, and the concept of energy dissipation for it is similar. The material of which the column is constructed is not important; it will be assumed to possess an elastoplastic load-deflection relation. The model, along with its load-deflection relation, is depicted in Figure A1.

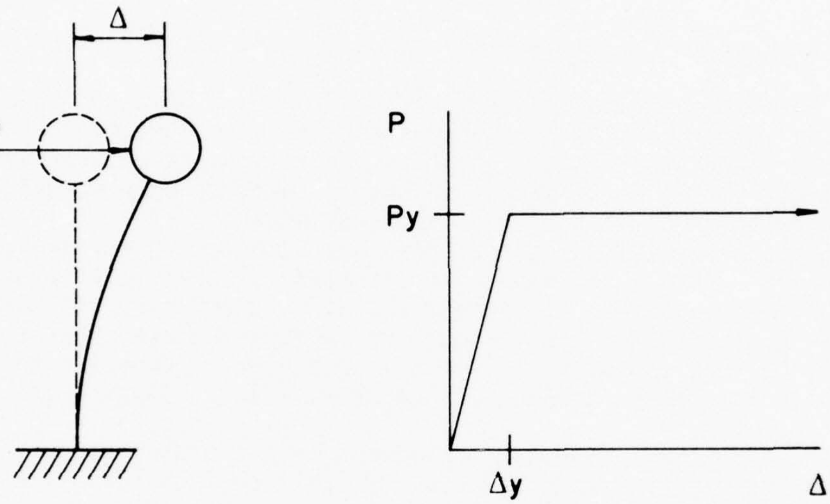

Figure A1. Single mass on elastoplastic column.

Monotonic Lateral Loading

The system is first assumed to be subjected to a slowly applied lateral load, as shown in Figure A1. If the magnitude of P remains less than P_y, the capacity of the column is not exceeded and response is linear. This is the traditional realm of structural analysis. If an attempt is made to increase P beyond P_y, the system will yield and the deflection will increase very rapidly. For a determinant, elastoplastic system, such as the one considered here, yield implies the collapse of the system. This is not the case, however, for an actual building. First, the force-deflection relation of an actual structural component is not elastoplastic. The slope of the force-deflection relation is not zero for deflections greater than Δy. Second, most actual structures possess redundancy. When one element yields, other elements which have not yielded carry load that cannot be carried by the yielded elements. Several elements must yield before the structure attains a collapse condition.

Seismic Loading

When the system is subjected to an earthquake, there is a rapid motion of its base. The mass is accelerated by forces transmitted through the column. The acceleration of the mass may be thought of as an inertial force, $m_o\ddot{x}$, applied to the column. In this way, a set of forces in the column would be defined. The concept is illustrated in Figure A2.

The base motion is generally a complicated function of time. The variation of ground acceleration with time measured at El Centro, California, during the 1940 Imperial Valley earthquake is shown in Figure A3. The velocity and displacement records, computed from the acceleration record by integration, are also plotted. A structure subjected to this motion would experience many loading reversals occurring at varying time intervals and at widely varying magnitudes of load, several of which may produce yielding.

For illustration, consider the column of Figure A2 to be subjected to several reversals of the inertial force of the mass, each producing the same maximum deflection. Figure A4 shows load-deflection relations. In Figure A4a, the system is subjected to a load of P_y and yields. When the deflection is equal to Δ_m (point c), the load is reversed. The system is unloaded and reaches zero load at point d. Upon loading in the opposite direction, the system follows line de, the path of loading being aimed at the yield point in the appropriate direction. Loading then continues until the system reaches point f. The deflection at point f is assumed to be equal to that at point c. The load is again reversed and the system reaches zero load at point g. Reloading of the system is now along the line gc. If the

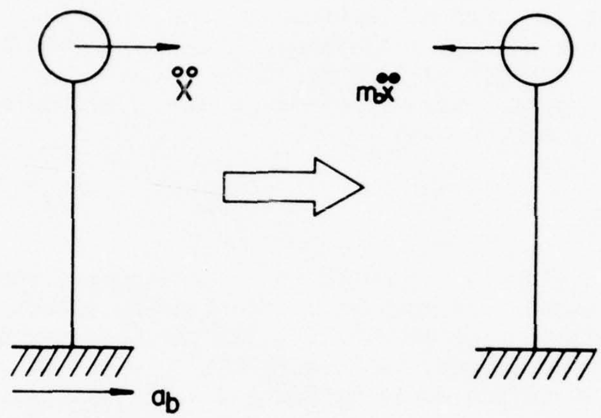

Figure A2. Earthquake loading on single mass and column.

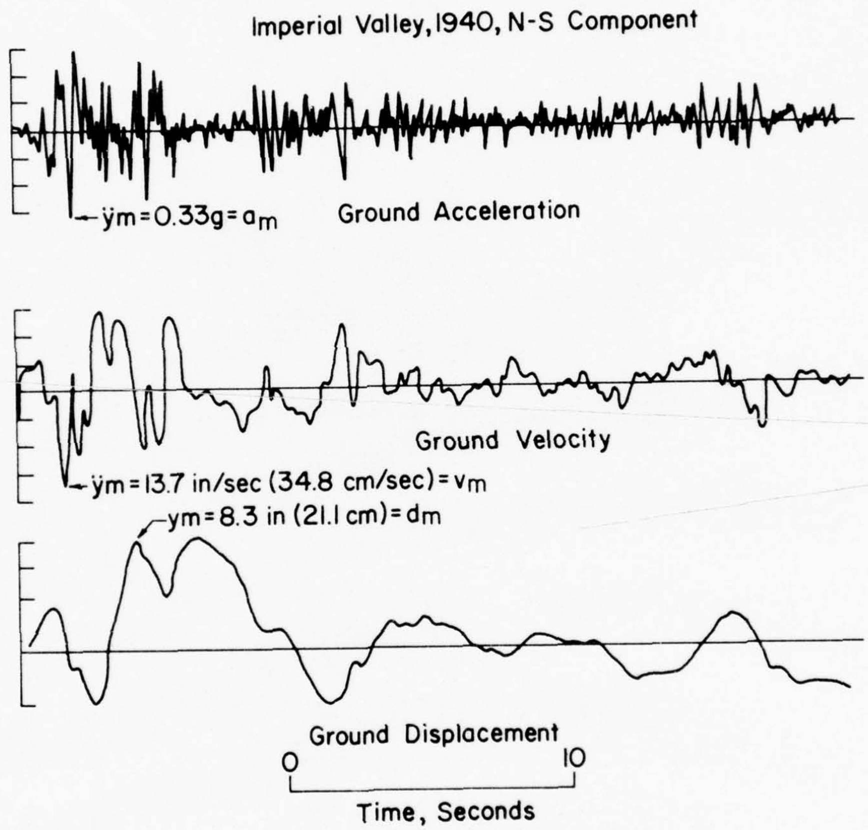

Figure A3. Sample earthquake base motion. From W. W. Hays et al., *Guidelines for Developing Design Earthquake Response Spectra*, Technical Report M-114/ADA012728 (U.S. Army Construction Engineering Research Laboratory [CERL], June 1975).

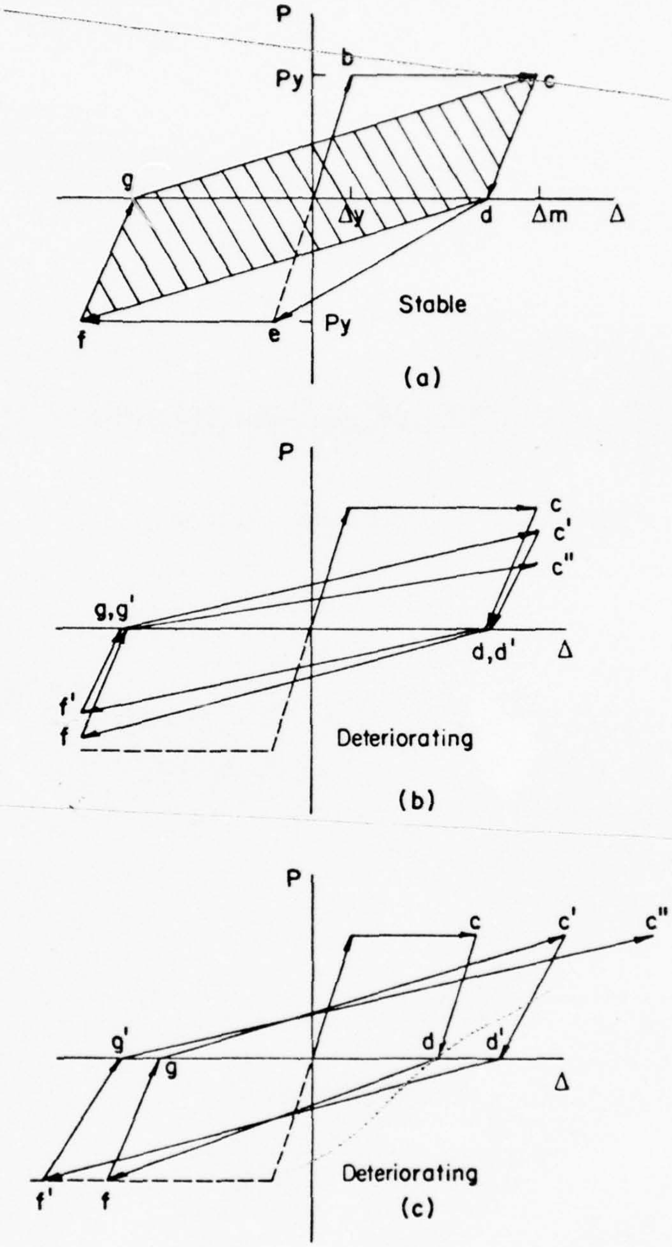

Figure A4. Comparison of stable and deteriorating hysteresis relations.

load is reversed again, the system will follow paths cd and df. Further reversal of load will cause the system to follow paths fg and gc.

The hysteresis just described is a highly idealized model[40] and is presented here to illustrate some important points. First, the first cycle is not typical of the load-deflection relation under repeated loading and unloading. Second, under repeated cycling to the same deflection limits, the system would continue to follow path cdfgc. This is referred to as a stable hysteresis. Under repeated cycling to the same deflection limits, the attained load remains the same. The area enclosed by the rectangle cdfg (the shaded area) is the energy dissipated by the system in one cycle of response. Note that the units of this area are force multiplied by deflection or energy. The ratio Δ_m/Δ_y (Figure A4a) is defined as the ductility attained by the system.

If the attained load decreases with successive cycles, when the system is cycled between a predetermined set of deflection limits, the hysteresis relation is said to deteriorate (Figure A4b). A deteriorating hysteresis generally represents structural failure. Successive cycles to the same deflection limits lead to a progressively lower load capacity. An additional case of deteriorating hysteresis, shown in Figure A4c, is characterized by the attainment of the same maximum load in successive cycles, but at progressively greater deflection limits. Hence, there must be a stable hysteresis relation if the system is to perform adequately under seismic loading. Often there is some maximum ductility at which the hysteresis relation is stable; for ductilities beyond this level, the hysteresis relation deteriorates. As a result, this maximum value of ductility represents a limiting parameter for inelastic response of the system.

A major effect of the stable energy dissipation phenomenon described above is that in the absence of energy input, i.e., base motion, the response will decrease with time, with the system eventually ceasing to move. Figure A5 shows a decreasing response in free vibration; in this case, if there were any energy input, the response would not decrease so sharply with time. If the rate of energy input were large enough, the response might even increase with time. The increase in response, however, would not be as great as it would be if there were no energy dissipation. Hence, given a specific rate of energy input, the greater the rate of energy dissipation, the lower the maximum system response will be.

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[40] T. Takeda, M. A. Sozen, and N. N. Nielsen, "Reinforced Concrete Response to Simulated Earthquakes," *ASCE Journal of the Structural Division*, Vol 96, ST12 (December 1970).

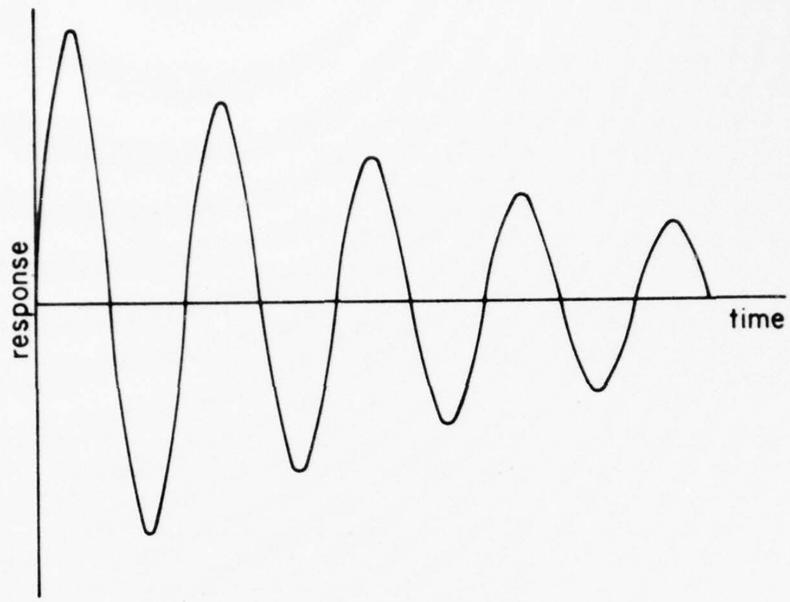

Figure A5. Effect of energy dissipation on free vibration response.

Hysteresis of a Reinforced Concrete Member

The system described in the previous section was very idealized. For an actual reinforced concrete member, the force-deflection relation is not elastoplastic and the hysteresis is not composed of linear portions. This section briefly describes the hysteresis properties of a reinforced concrete member. For additional information, the reader may refer to two papers from which much of the following material is taken.[41,42]

Figure A6 shows the member being considered and its hysteresis relation. The member is a doubly reinforced, single column; the two layers of reinforcement contain equal areas of steel. The behavior considered will be flexural. Possible failures in shear or failures

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[41] B. Bresler, *Behavior of Structural Elements - A Review*, Building Science Series 46 (National Bureau of Standards, February 1973).
[42] M. A. Sozen, "Hysteresis in Structural Elements," *Applied Mechanics in Earthquake Engineering* (American Society of Mechanical Engineers, 1974).

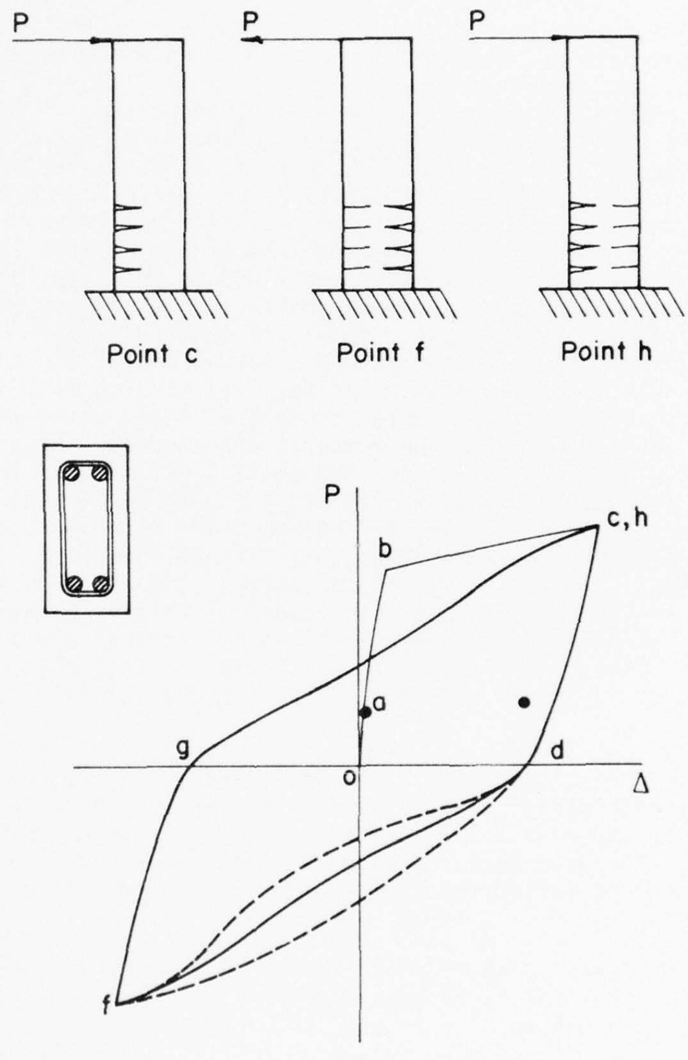

Figure A6. Hysteresis for reinforced concrete.

of bond between concrete and steel will be ignored. The column is assumed to be fixed at its base, and problems concerned with anchorage of the steel at the base will be ignored.

As the column is loaded along path oa, beginning at point o, the force-deflection relation is linear; the stress strain relations of both steel and concrete are linear for this low level of loading. At point a, the concrete on the tension face of the column begins to crack, resulting in a decrease in stiffness. As the column is loaded along path ab, the cracks increase in size and number, and the compressive stress on concrete at the compression face of the column increases. At point b, the steel layer acting in tension reaches its yield stress, resulting in an abrupt decrease in the column's stiffness. The shape of the load-deflection relation between points b and c is a complicated function of the particular nature of the steel and concrete stress-strain relations, the steel arrangement and ratio, and the overall section geometry. At point c, the load is reversed. The force-deflection relation for the unloading has a fairly steep slope, reflecting the steep slopes of the stress-strain relations for steel and concrete. The nature of the load-deflection relation between points d and f is a function of the material stress-strain relations, the closing of the cracks from the first one-quarter cycle (those cracks now being adjacent to the compression face of the member), the opening of new cracks in tension, the steel ratio, and the overall geometry of the section. Depending on these numerous variables, the path of loading may be similar to the solid path between points d and f, or perhaps, more similar to either of the dashed paths. The well-defined yield, characterized by point b, is not observed in the hysteresis after the first one-quarter cycle of the loading. This is because the stress-strain relation of the steel does not exhibit a well-defined yield point after the first one-quarter cycle. This phenomenon, in the steel stress-strain relation, is referred to as the Bauschinger Effect.

When the loading sequence reaches point f, the section is cracked symmetrically. The path of unloading and reloading through points f, g, and h will be similar to the path through points c, d, and f. If the hysteresis relation is stable and the column is cycled between the same deflection limits, the loop cdfgh will be repeated indefinitely.

This hysteresis relation is for flexural behavior. Problems such as failure of the column in shear, failure of the bond between steel and concrete, and lack of confinement for the concrete may cause a deterioration of the hysteresis relation. It should also be emphasized that this relation is for one element. The hysteresis relation for the entire structure is a function of the hysteresis relations of its many elements. Finally, the relation is for a fairly

long, slender member with only two layers of steel. Alterations would occur for deeper members, such as walls, or for members with numerous steel layers.

Idealization With an Elastic System

The general concept behind the evaluation procedure is to model the column of Figure A2 with its hysteretic response as the familiar linear spring and viscous dashpot model, shown in Figure A7. This enables the maximum response to be computed from a linear response spectrum. The problem is to determine a spring stiffness and a damping factor such that the response of the linear system will be equal to that of the hysteretic system. An undamped system with a stiffness, k, equal to k_{unc} would generally have an acceleration response much higher than that for the hysteretic system. The reduction in response may be thought of as caused by two factors. First, response is reduced because of the decrease in stiffness or frequency that accompanies inelastic action. Second, response is reduced because of the energy dissipated by the hysteresis loop abcd. One approach is to consider a reduced stiffness k_μ, where k_μ is given by k_{unc}/μ along with a damping factor β, such that the energy dissipated by the viscous dashpot is equal to that dissipated by the hysteresis loop. Such approaches are discussed in several publications.[43-46]

The above concept is applied to the evaluation procedure in a somewhat reversed form. The initial, or undamped, stiffness of the spring, k_{unc}, is determined. The initial system frequency, f_{unc}, is computed from

$$f_{unc} = \frac{1}{2\pi}\sqrt{\frac{k_{unc}}{m_o}} \qquad \text{[Eq A1]}$$

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[43] A. Shibata and M. A. Sozen, *The Substitute Structure Method for Earthquake Resistant Design of Reinforced Concrete Frames*, Structural Research Series No. 412 (University of Illinois, October 1974).

[44] P. Gulkan and M. A. Sozen, *Response and Energy Dissipation of Reinforced Concrete Frames Subjected to Strong Base Motions*, Structural Research Series No. 377 (University of Illinois, May 1971).

[45] L. S. Jacobsen, "Steady Forced Vibration as Influenced by Damping," *Transactions ASME*, Vol 52, Part 1 (ASME, 1930).

[46] P. C. Jennings, "Equivalent Viscous Damping for Yielding Structures," *ASCE Journal of the Engineering Mechanics Division*, Vol 94, EM1 (February 1968).

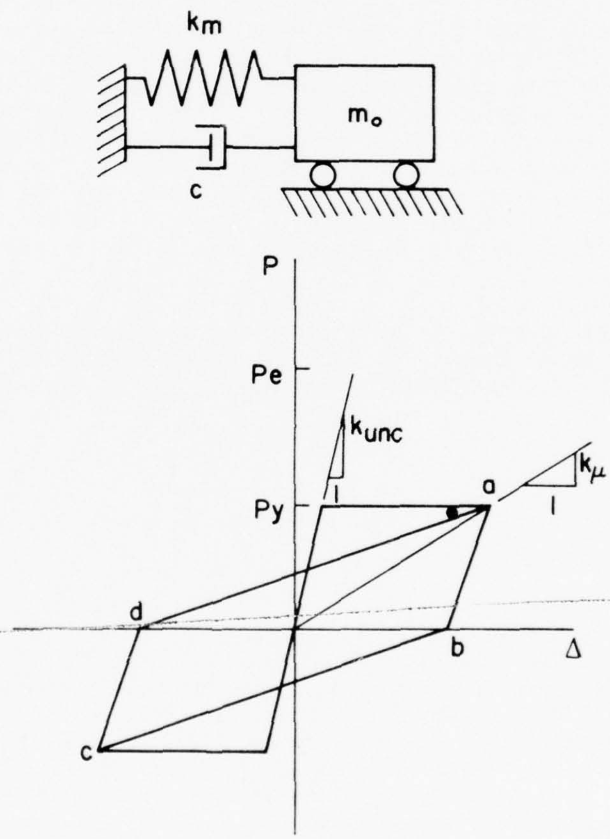

Figure A7. Modeling of nonlinearity
and energy dissipation.

where m_0 = concentrated mass of a single-degree-of-freedom system. The spring-mass-viscous-dashpot system constitutes a single-degree-of-freedom system. For a given earthquake record, assuming linearly elastic response, the maximum response may be computed knowing only the frequency, f, and the damping factor, β, where,

$$\beta = \frac{c}{2\sqrt{km_0}} \qquad \text{[Eq A2]}$$

and k = spring stiffness for a single-degree-of-freedom system. Using a frequency equal to f_{unc} and an arbitrarily chosen damping factor, β_s, the maximum absolute acceleration, $\ddot{s}$, of the system is determined. To accomplish this, a previously prepared plot of the variation of maximum response with frequency for a given damping factor is generally used. This relation is a specific for the earthquake record chosen, and is referred to as a response spectrum for the earthquake. Next, the total inertial force on the mass would be obtained from

$$P_e = m_0\ddot{s} \qquad \text{[Eq A3]}$$

This represents the force level in the system consistent with linearly elastic response. If the load-deflection relation for the spring is idealized as elastoplastic, as in Figure A7, the strength of the system is given by the load, P_y. The value of P_y is calculated. The disparity between the computed values of P_e and P_y represents the energy dissipation that must occur in the system. The idealization of Figure A7 represents this energy dissipation in two parts--spring stiffness reduction from k_{unc} to k_μ and viscous damping action of the dashpot in the model. Analytical studies[47-49] have produced some very approximate rules for the force reduction produced by these two factors. The stiffness reduction is expressed in terms of required ductility, μ. The viscous damping is expressed in terms of damping factor, β. The rules are used to delineate various combinations of damping and ductility that could account for the disparity between P_e and P_y.

[47] N. M. Newmark and W. J. Hall, *Procedures and Criteria for Earthquake Resistant Design*, Building Science Series 46 (National Bureau of Standards, February 1973).

[48] N. M. Newmark and W. J. Hall, "A Rational Approach to Seismic Design Standards for Structures," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 2 (1974).

[49] N. M. Newmark, "Current Trends in the Seismic Analysis and Design of High-Rise Structures," *Earthquake Engineering*, R. L. Wiegel, Coordinating Editor (Prentice-Hall, Inc., 1970).

There will be a whole array of ductilities and damping factors that would reduce the system response to the required level. In a crude sense, some damping can be replaced by some additional ductility. At any rate, the final step is to decide whether the structure is capable of providing the required force reduction through inelastic action. It must be decided if any of the combinations of damping and ductility determined above are within the capacity of the system.

This discussion has been for a system with one degree of freedom--a single mass with a single vertical element. An actual structure, however, has several masses and several vertical elements. Appendix C describes how this concept is extended to such a system.

APPENDIX B:

NONDUCTILE STRUCTURAL PROBLEMS

Appendix A emphasized the significance of a structure's capacity for ductility as well as the necessity for strength. Chapter 2 pointed out that various structural systems and failure mechanisms for each system exhibit different capacities for ductility. There are several structural behavior phenomena that could drastically affect the ductility of the mechanisms described in Chapter 2. These include phenomena such as loss of steel-concrete bond, shear failures in flexural members, failures in beam-column or slab-column joints, and loss of anchorage for steel.

In many cases, the proper evaluation of many of these problems depends on the availability of detailed blueprints. It also requires that the structure conform to those prints in many detailed aspects. These requirements will often make checking many of the phenomena affecting ductility impossible. In other cases, the engineer may skip several of these checks for structures that are not so critical to the facility's function or whose failure would not create great life hazard. In still other cases, these checks may be skipped if more basic evaluations produce very conclusive data for evaluating the structure. However, the engineer should be aware that these failures may lead to a steady degradation of the structure by greatly decreasing ductility and that some cases may result in complete collapse of the structure.

Shear Failure in Flexural Members

In the discussion of the ductility of reinforced concrete frames in Chapter 2, the failure mechanism was assumed to be characterized by flexural hinging of the members. However, the members could also fail in shear, which would drastically reduce the member's capacity

for ductility. Shear failure is characterized by deterioration of strength with repeated cycling (unstable hysteresis) and eventually results in total loss of the member. The importance of shear failure[50,51] and examples of such failures are discussed in a number of publications.[52-56]

Figure B1 illustrates the shear failure mechanism for a member. The mechanism is described in greater detail in a University of Illinois publication[57] from which much of this material is taken. The illustration at the top of the figure depicts a cantilevered flexural member loaded at its free end by a cyclic load, P. The member's longitudinal and transverse reinforcement are shown in the drawing. Two vertical dashed lines delineate a portion of the member which is enlarged in portions a through d of the figure, which illustrate the progressive stages of the shear failure. In an early cycle, shear stress carried in the concrete results in the symmetrical diagonal crack pattern of part a. Repeated cycling causes the sides of the cracks to move relative to one another, slowly grinding away concrete and making the cracks wider. Eventually the member behavior is characterized as a system of concrete boulders held together by the transverse steel (part b of Figure B1). All shear (diagonal

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[50] G. V. Berg and R. D. Hanson, "Engineering Lessons Taught by Earthquakes," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[51] B. Bresler, *Behavior of Structural Elements - A Review*, Building Science Series 46 (National Bureau of Standards, February 1974).

[52] S. Okamoto, *Introduction to Earthquake Engineering* (John Wiley & Sons, 1973).

[53] H. S. Lew, E. V. Leyendecker, and R. D. Dikkers, *Engineering Aspects of the 1971 San Fernando Earthquake*, Building Science Series 40 (National Bureau of Standards, December 1971).

[54] A. K. Chopra, V. V. Bertero, and S. A. Mahin, "Response of Olive View Medical Center Main Building During San Fernando Earthquake," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[55] L. G. Selna, M. D. Cho, and R. K. Ramanathan, "Evaluation of Olive View Hospital Behavior on Earthquake Resistant Design," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[56] V. V. Bertero, B. Bresler, L. G. Selna, A. K. Chopra, and A. V. Koretsky, "Design Implications of Damage Observed in Olive View Medical Center Buildings," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[57] J. K. Wight and M. A. Sozen, *Shear Strength Decay in Reinforced Concrete Columns Subjected to Large Deflection Reversals*, Structural Research Series No. 403 (University of Illinois, August 1973).

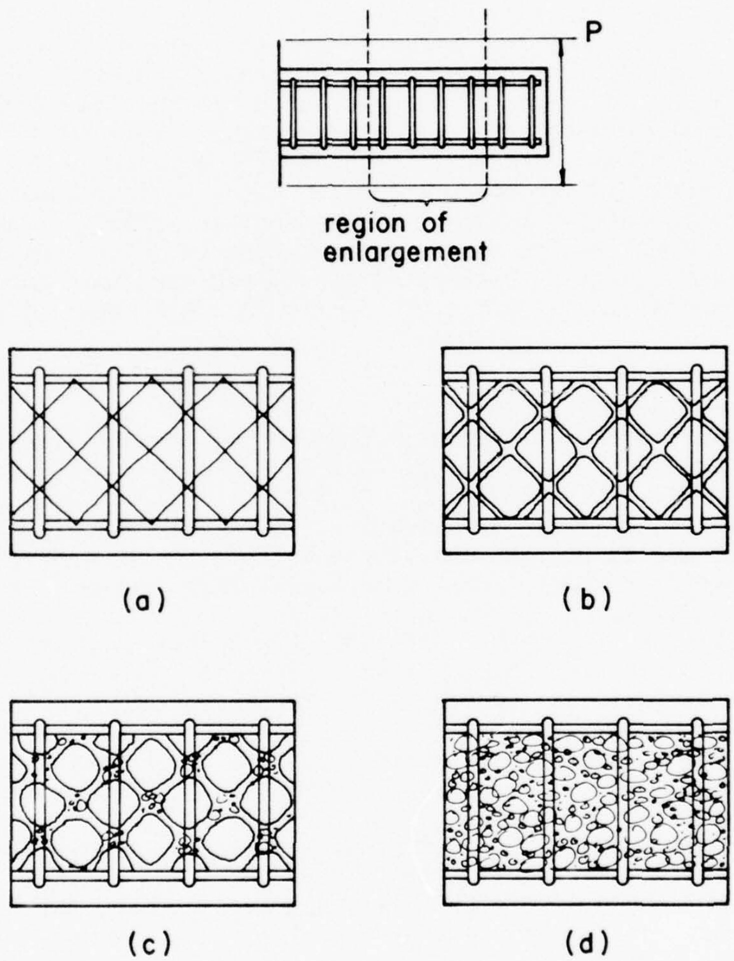

Figure B1. Deterioration of reinforced concrete section in shear with cycling.

tension) must be resisted by the transverse reinforcement at this state. The concrete, which can act only in compression, provides bearing for the transverse steel system. As cycling continues, so does grinding away of the concrete (part c). The concrete boulders become progressively rounder and eventually lose the ability to provide bearing for the transverse steel. The result is a characteristic degradation in the member's stiffness and strength.

If the above failure pattern is to be avoided, diagonal cracking of the concrete must be controlled or even prevented entirely. Bresler, and Wight and Sozen assert that for design purposes, no shear should be required to be carried by the concrete,[58,59] and that the total shear force on the section should be assigned to the transverse reinforcement. It should also be noted that the combination of high axial load, moment, and shear may cause shear failure to be more likely. Such members should be evaluated carefully.

The likelihood of shear failure may be especially high if the effective column length is very short. The spandrel wall assembly shown in Figure B2 is an example of such a situation. (For examples of short column failure, see the publications referenced below.[60-62]) Failures of short columns, such as in the spandrel wall assembly, may be characterized by a mechanism even more brittle than the one described above. An experimental study has been reported in which failure was obtained by fracture of the transverse reinforcement;[63] results of this study suggest that the shear force assigned to concrete be zero and that shear forces in the stirrups be computed

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[58] B. Bresler, *Behavior of Structural Elements - A Review*, Building Science Series 46 (National Bureau of Standards, February 1973).

[59] J. K. Wight and M. A. Sozen, *Shear Strength Decay in Reinforced Concrete Columns Subjected to Large Deflection Reversals*, Structural Research Series No. 403 (University of Illinois, August 1973).

[60] A. R. Flores, "The Luzon Earthquakes of August 2, 1968 and April 7, 1970," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[61] G. V. Berg and R. Husid, "Structural Behavior in the 1970 Peru Earthquake," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[62] S. S. Tezcan, M. Ipek, and S. Acar, "Reasons for the Earthquake Damage to the New High School Building in Burdur, Turkey," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974),

[63] J. G. Bouwkamp and Ö. Küstü, "Experimental Studies of Spandrel Wall Assemblies," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

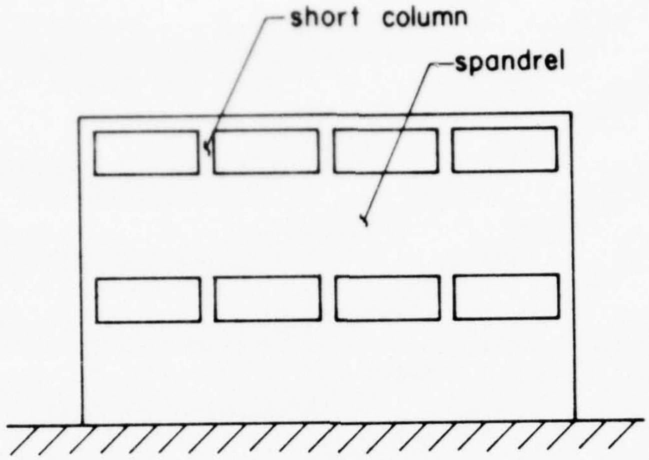

Figure B2. Spandrel wall assembly.

from a 45° inclined crack and the truss analogy, as suggested by Bresler and by Wight and Sozen, for beams and columns of more usual proportions. However, the study also suggests that the minimum stirrup spacing be given by d/3, where d is the effective depth of the section. Note that this is more stringent that the requirement of the ACI code. Section 11.14 of that standard fixes the minimum stirrup spacing at d/2, where d is as defined previously.

Beam-Column Joints

Previous discussions of reinforced concrete frames assumed that failure was characterized by flexural hinging at the member-joint interface. However, there is also a possibility of failure within a joint, which is often related to failure in shear or loss of steel-concrete bond. The problem is recognized in several studies.[64-66]

[64] G. V. Berg and R. D. Hanson, "Engineering Lessons Taught by Earthquakes," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[65] R. Park and T. Paulay, "Behavior of Reinforced Concrete External Beam-Column Joints Under Cyclic Loading," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[66] B. Bresler, *Behavior of Structural Elements - A Review*, Building Science Series 46 (National Bureau of Standards, February 1974).

The ACI Code provides explicit provisions to avoid joint failure.

Figure B3 illustrates a portion of a reinforced concrete frame. The figure shows a portion of the frame consisting of a joint core, and one-half of the length of each of the four members framing into the joint with the entire system's applied forces. The boundaries of the separated portion are delineated by the dashed rectangle in the upper portion of the figure. The figure points out the change in the character of the joint loading caused by loading the frame laterally rather than vertically. These two loading patterns are contrasted in the lower portion of the figure. For the case of vertical loading only, the moment and the vertical force on the two sides of the joint are in the same direction. For the case of lateral loading, they are in opposite directions. Hence, lateral loading imposes more severe deformation requirements on the joint core. The fact that the lateral loads are cyclic further aggravates the problem. The deformations reverse in sense during each cycle. In many older structures, this problem may not be recognized in the design.

Figure B4 illustrates a joint core subjected to shearing forces. As the shear in the joint is cycled, diagonal tension cracks may form--first in one direction, then in the other--to form a symmetrical crack pattern, as shown in the figure. Transverse steel is needed in the joint, both to resist the cyclic shear force and to control the width of the diagonal cracks. This prevents grinding away of the core concrete by the repeated reversal of shear and bending deformation, and prevents expansion of core concrete that could decrease material strength. If the transverse steel is to accomplish these objectives, it must not yield under cyclic loading. The consequence of such a deterioration in shear is a severe loss in joint shear stiffness accompanied by a significant decrease in overall structure stiffness. Researchers have found that this may exert far more than a second-order effect on structural stiffness.[67]

A failure within the joint may also be characterized by loss of anchorage to the flexural reinforcement of the members framing into the joint. Bond deterioration of this sort under cyclic loading appears to be caused by crushing of the concrete adjacent to the bar lugs. If the joint is external with columns framing above and below, the bond deterioration may be more severe for horizontal bars in the top portion of a joint or for outer column reinforcement.[68]

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[67] R. Park and T. Paulay, "Behavior of Reinforced Concrete External Beam-Column Joints Under Cyclic Loading," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[68] R. Park and T. Paulay.

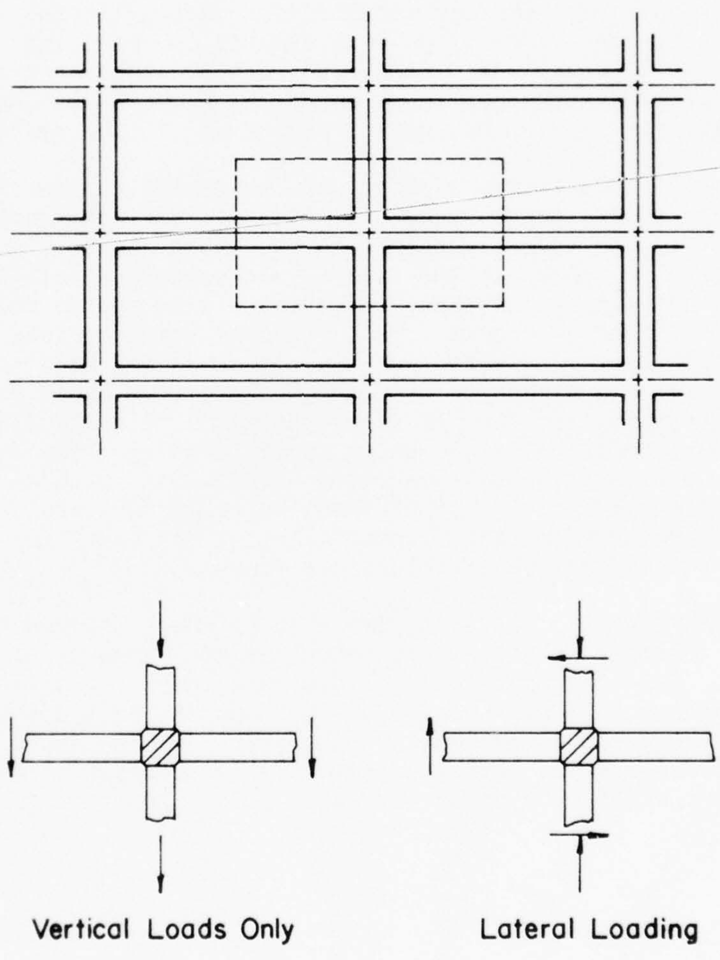

Figure B3. Joint loading for vertical and lateral loads.

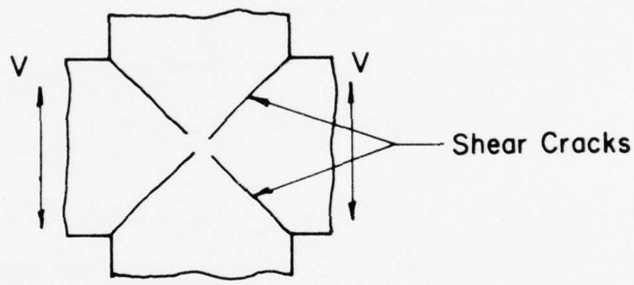

Figure B4. Shear cracks in joint core.

It has been suggested that the anchorage length for horizontal beam reinforcement should be calculated between the end of the bar and the point at which it is either bent upward or downward; i.e., bond along the horizontal portion of the bar is ignored.[69] It should be noted that the confining effect of transverse steel can again help retard an anchorage failure. It can increase the concrete strength, discouraging the crushing and splitting of concrete that characterize loss of bond. Like shear failure, anchorage failure may lead to a drastic reduction in structure stiffness. The structure may become a mechanism with flexible joints.[70] Note that, like shear failures, anchorage failure is encouraged when the reinforcement is actually stronger than the engineer believes it is. Higher steel strength may increase the moment capacity of the joint-member interface. The load for anchorage failure, however, does not increase. Thus anchorage failure may occur rather than flexural failure.

Additional beams framing into a joint laterally help to retard both nonductile failures mentioned above.[71,72]

In conclusion, the ultimate strength should be characterized by the condition of Figure B5, characterized by yield of the flexural reinforcement accompanied by compression in concrete.

Longitudinal Steel Ratio

A ductile flexural failure requires that when the section reaches its maximum moment, the tension reinforcement yields before the concrete crushes in compression. This is described in standard texts on reinforced concrete.[73] The ACI Code requires that the longitudinal steel ratio, p, be less than or equal to 0.75 times the balanced ratio (simultaneous yield of steel and crushing of concrete), p_b. However, it has been suggested that in regions of very high ductility requirements (hinge regions), the maximum steel ratio should be reduced. The suggested value is $p \leq 0.5\ p_b$.[74] At any rate, the engineer may

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[69] R. Park and T. Paulay, "Behavior of Reinforced Concrete External Beam-Column Joints Under Cyclic Loading," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[70] B. Bresler, *Behavior of Structural Elements - A Review*, Building Science Series 46 (National Bureau of Standards, February 1974).

[71] B. Bresler.

[72] R. Park and T. Paulay.

[73] P. M. Ferguson, *Reinforced Concrete Fundamentals*, Second Ed. (John Wiley & Sons, 1965).

[74] B. Bresler.

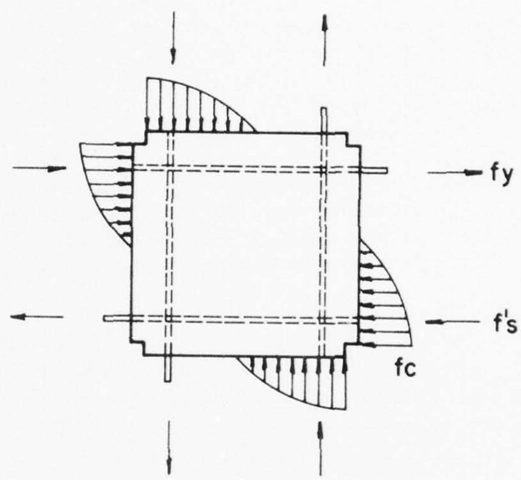

Figure B5. Failure mechanism for beam-column joint.

wish to compute the longitudinal steel ratio and compare it to the balanced ratio. If the steel ratio is very close to 0.75 times the balanced ratio, he should note that this can affect structure ductility.

Confinement of Concrete

If the reinforced concrete section is to endure repeated cycling at high strain levels, the core concrete must be well-confined by the transverse reinforcement. Dangers of lack of confinement for core concrete are illustrated in several publications.[75-80] Deficiencies could occur either in the characteristics of the confining devices or in the spacing of the devices. Buildings designed under older codes should especially be checked for confinement deficiencies. Unfortunately, such a check is almost entirely dependent on the presence of extensive blueprints. Even when blueprints are present, the building may not conform well with the plans. However, where blueprints are available, the engineer should try to develop a general opinion concerning the possible adequacy of confinement.

In general, spirals are superior to hoops, although hoops will be more prevalent. The spacing of hoops would have to be unrealistically close to equal the confining ability of a spiral.[81] Tests have

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[75] A. K. Chopra, V. V. Bertero, and S. A. Mahin, "Response of Olive View Medical Center Main Building During San Fernando Earthquake," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[76] L. G. Selna, M. D. Cho, and R. K. Ramanathan, "Evaluation of Olive View Hospital Behavior on Earthquake Resistant Design," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[77] V. V. Bertero, B. Bresler, L. G. Selna, A. K. Chopra, and A. V. Koretsky, "Design Implications of Damage Observed in Olive View Medical Center Buildings," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[78] H. S. Lew, E. V. Leyendecker, and R. D. Dikkers, *Engineering Aspects of the 1971 San Fernando Earthquake*, Building Science Series 40 (National Bureau of Standards, December 1971).

[79] A. R. Flores, "The Luzon Earthquakes of August 2, 1968 and April 7, 1970, *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[80] S. S. Tezcan, M. Ipek, and S. Acar, "Reasons for the Earthquake Damage to the New High School Building in Burdur, Turkey," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[81] B. Bresler, *Behavior of Structural Elements - A Review*, Building Science Series 46 (National Bureau of Standards, February 1973).

been reported in which columns equipped with spiral reinforcement were subjected to cyclic loading far into the inelastic range. The columns were tested to the stage where the confined core concrete had completely crumbled, yet the resisting moment did not decline. The hysteresis relation was stable. The column owed its success to the confining ability of the spiral.[82]

Hoops appear to be inferior to spirals for several reasons.[83] The confining stress is dependent on the hoop tension, which obtains its reaction from bearing stresses concentrated at the corners of the hoop. These stresses may limit confinement. Furthermore, the core may bulge laterally in the interval between the hoops. This bulging causes tension and accompanying cracking in the concrete near the hoops, thus reducing core area.

However, hoops will be the most frequently encountered case, and the engineer must evaluate their adequacy. At this time, the most likely guide is probably Appendix A (Section A.6.4) of the ACI Code. The minimum spacings published in that section are derived to obtain a confining stress equal to that of a spiral of the same core area in a column of equal gross area, with the same center-to-center spacing of transverse reinforcement and equal transverse reinforcement strength. Further allowance is made for the hoops being only 50 percent as efficient in inducing confining stress as the spiral. The result is believed to be conservative.[84]

Additional Steel Detailing Problems

Problems may be caused by splices in the main longitudinal, or flexural, reinforcement. The steel may not develop its full strength, and may fail brittly at the splice. This can be especially dangerous in regions of flexural hinging, where the steel may need to develop its ultimate strength, as opposed to yield strength, if the full ductility of the failure mechanism is to be realized. The engineer might examine the blueprints for evidence of splicing in hinging regions. The ACI Code (Section A.6.7) provides guidance concerning splice lengths.

Structural walls should be provided with certain minimum ratios of horizontal and vertical reinforcement. Section A.8 of the ACI Code recommends values.

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[82] B. I. Karlsson, H. Aoyama, and M. A. Sozen,"Spirally Reinforced Concrete Columns Subjected to Loading Reversals Simulating Earthquake Effects," *Proceedings Fifth World Conference on Earthquake Engineering* (1974).

[83] B. Bresler, *Behavior of Structural Elements - A Review*, Building Science Series 46 (National Bureau of Standards, February 1973).

[84] *Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-71)* (American Concrete Institute, 1971).

As mentioned previously, openings in a structural wall may generate stress concentration problems at the corners. In addition, bending stresses may be high throughout the perimeter of the opening (Figure B6). There should be additional reinforcement concentrated around the perimeter of an opening, as discussed in TM 5-809-10.

It is also important to have a continuity of reinforcement where two structural walls intersect, i.e., at a corner of a building. There have been failures characterized by separation of walls at such a detail.[85] Figure B7 illustrates the contrast between continuity and its absence.

Ductility of a flexural member may also be limited by buckling of the longitudinal reinforcement under compressive deformation. The result is loss of moment capacity and corresponding degradation in the hysteresis relation.[86] The arrangement of the transverse reinforcement, i.e., the hoops, largely determines the susceptibility of the longitudinal reinforcement to buckling. Among the important characteristics are the size of the hoops, the spacing of the hoops, and the anchorage of the hoop ends. The spacing determines the unsupported length of a "column" of reinforcement, and the size and anchorage determine the end restraint conditions for that "column." The problem of avoiding buckling is really that of insuring that the yield stress of the reinforcement in compression is less than the critical inelastic buckling stress of the reinforcement "column." Bresler[87] suggests a formula for the maximum center-to-center distance between adjacent ties if buckling of the longitudinal reinforcement is to be avoided:

$$\frac{s_m}{D} = B\sqrt{\frac{E_t}{f_y}} \qquad \text{[Eq B1]}$$

where s_m = maximum center-to-center distance between ties
D = diameter of longitudinal steel
E_t = tangent modulus of longitudinal steel at yield strength based on nonlinear stress-strain relation
$B \approx 2.2$.

[85] L. W. Bockemohle, "Earthquake Behavior of Commercial-Industrial Buildings in the San Fernando Valley," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974)
[86] B. Bresler, *Behavior of Structural Elements - A Review*, Building Science Series 46 (National Bureau of Stnadards, February 1973).
[87] B. Bresler.

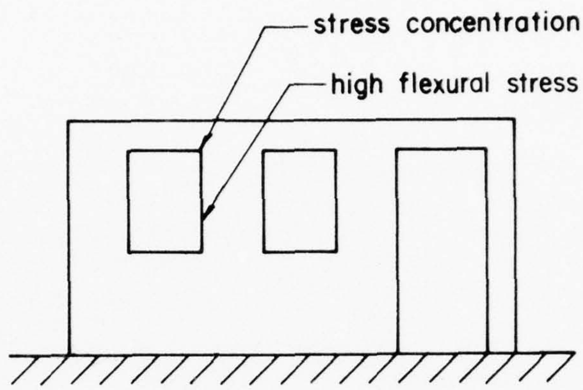

Figure B6. Problems around openings in a wall.

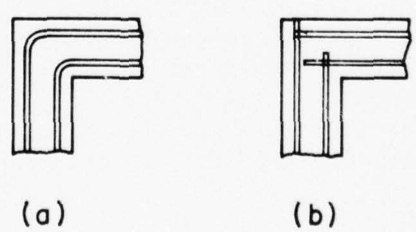

Figure B7. Corner problem in wall system.

Bresler suggests that under cyclic loading, an E_t value of 2000 ksi (14 000 MPa) might be appropriate for an f_y value of 50 ksi (345 MPa). In this case, s might vary from 7 to 5 diameters. At any rate, the engineer should consider the general magnitude of the hoop spacing in relation to this problem.

APPENDIX C:

STRUCTURAL RESPONSE

In the past, most seismic design codes have assumed elastic response, with static, lateral forces as design loads. The lateral loads have been assumed to vary linearly with height. Structure ductility has been accounted for by reducing the lateral forces below those which would correspond to elastic response.

With the development of modal analysis computer programs and response spectrum techniques, it is now becoming practical to more explicitly account for a structure's dynamic response, especially for more important or critical structures. This includes calculating a more characteristic mode shape, or variation of acceleration with height, rather than using an assumption of linear variation. It also includes the use of a design spectrum developed for the site of the building being evaluated. This procedure is a means of considering site seismicity and the variation of structure response with structure natural period more explicitly. Finally, these results may be explicitly adjusted to account for the damping and ductility capacities of various types of structural systems. This adjustment is embodied in an energy dissipation factor. This method is illustrated in Figure C1. Another method assumes a linear mode shape and provides a standard relation between response amplification and natural period. It still, however, provides an explicit adjustment for the damping and ductility capacity of various types of structural systems, i.e., the energy dissipation factors are still used.

A modal analysis response spectrum energy dissipation approach not only provides a more accurate characterization of a complex structure's dynamic response, but it better allows for the independent modification of each portion of the design or evaluation procedure as structural performance is observed and new research results become available.

The present evaluation procedure is illustrated in Figure C2. A modal analysis is performed on the structure and a design spectrum is used to obtain a set of linearly elastic force requirements. The as-built structure capacity is then estimated by determining the yield

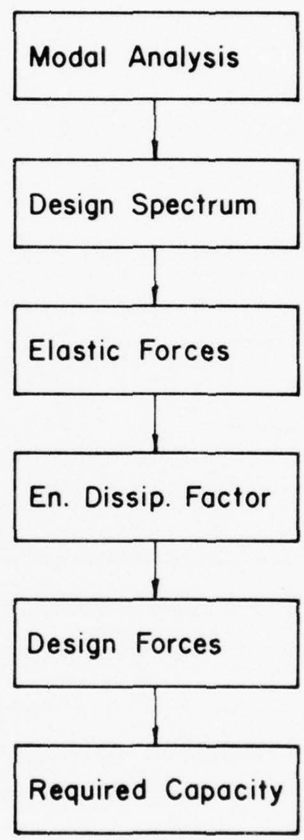

Figure C1. Damping and ductility design approach.

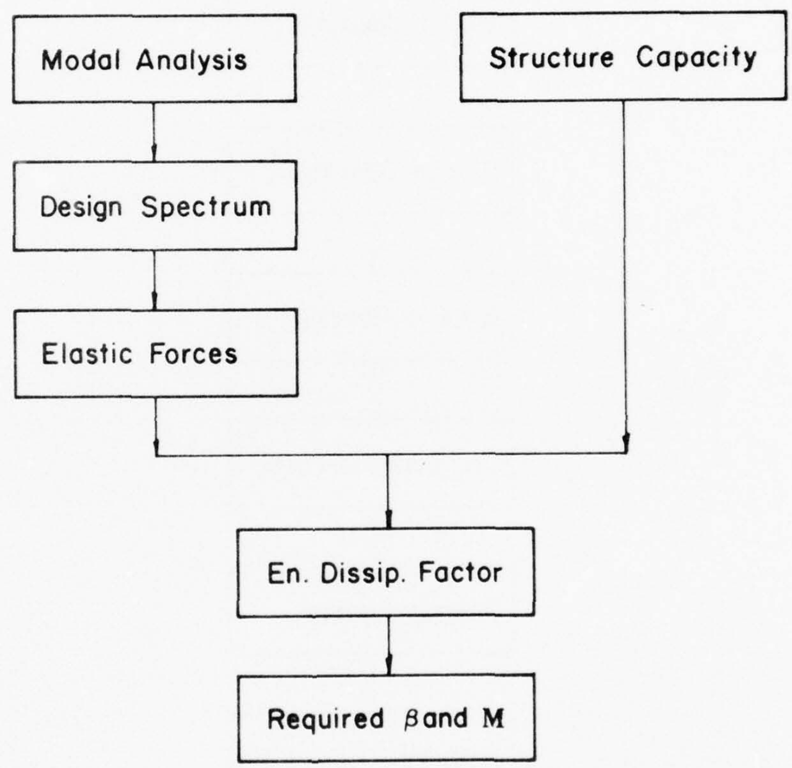

Figure C2. Evaluation approach.

mechanism. The elastic force requirements and the capacity are compared to obtain an energy dissipation factor. Various quantities of damping, β, and ductility, μ, that imply the required energy dissipation factor are considered. These are compared to the estimated capacities for damping and ductility of the structure being evaluated.

Modal Analysis Approach

The modal analysis approach to dynamic response of structures is described in detail by Stockdale[88] and Biggs.[89] A brief explanation of the application of the method to the present problem is provided below.

Single-Degree-of-Freedom System

Figure C3 illustrates the simple mass-spring system discussed in Appendix A, referred to as a linear single-degree of freedom system (SDF); the mass is capable of only one type of motion, in this case, horizontal displacement. The equation of motion of the system is given by:

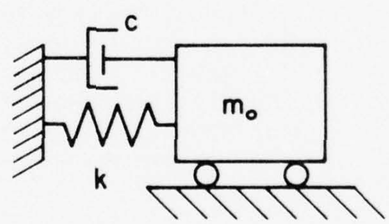

Figure C3. Single-degree-of-freedom system.

[88] W. K. Stockdale, *Modal Analysis Methods in Seismic Design for Buildings*, Technical Report M-132/ADA012732 (CERL, June 1975).
[89] J. M. Biggs, *Introduction to Structural Dynamics* (McGraw-Hill Book Co., 1964).

$$m_o\ddot{u} + c\dot{u} + ku = m_o\ddot{x}_b \qquad \text{[Eq C1]}$$

where m_o = mass
c = viscous damping coefficient
k = spring stiffness
u = displacement of the mass relative to the base (a function of time)
$\ddot{x}_b$ = base acceleration (a function of time)
$\dot{u}$ = velocity of the mass relative to the base
$\ddot{u}$ = acceleration of the mass relative to the base.

The response of this SDF system is characterized by a certain natural frequency of cyclic motion, given by

$$f = \frac{1}{2\pi}\sqrt{\frac{k}{m}} \qquad \text{[Eq C2]}$$

where f = natural frequency.

Indeed, the SDF system is totally defined given the frequency, f, and a damping factor, β, given by

$$\beta = \frac{c}{2\sqrt{km_o}} \qquad \text{[Eq C3]}$$

Knowing these two parameters, one need only know the base motion to compute the response of the SDF system. Given the variation of base acceleration with time, the response history for the mass could be computed. Displacement, velocity, or acceleration could be determined. However, for most work, it is expensive and unnecessary to compute the response for all times; the maxima themselves are sufficient. For a given base motion, the maximum responses may be plotted as functions of frequency and damping. The maximum responses for an array of SDF systems are represented (a response spectrum).

The obvious question concerns what base motion to use. For each facility site, certain geologic data, along with idealizations of response spectra computed from measured earthquakes, have been used to construct a design spectrum. This spectrum will be provided for some arbitrary value of damping factor and should be used for calculations. From this, the engineer may read the maximum displacement relative to the base, maximum velocity relative to the base, or maximum absolute acceleration for an SDF system of a given natural frequency and damping factor.

Several Degrees of Freedom

An actual building is more xomplex than the simple system described above. There would be several masses or floors, and each might exhibit several types of motion. There would be many degrees of freedom. In the system of Figure C4, there are three floors of masses m_1, m_2, and m_3, connected by lateral load-resisting elements with moments of inertia I_1', I_2', I_3' and areas A_1', A_2', and A_3'. In a general case, for a planar or two-dimensional idealization, as is being discussed here, each mass may exhibit three degrees of freedom. These consist of horizontal displacements Q_1, Q_2, and Q_3, vertical displacements Q_4, Q_5, and Q_6, and rotations Q_7, Q_8, and Q_9, as shown in Figure C4--a total of nine degrees of freedom. The modal analysis response spectrum approach provides the engineer with a means of expressing the response of a multi-degree-of-freedom system as a combination of the responses of several SDF systems.

Only those degrees of freedom associated with a significant effect on the mode shapes or natural frequencies need to be considered. The vertical degrees of freedom (Q_4, Q_5, and Q_6) would, in most cases, affect modes primarily associated with vertical motion. Such motion is generally not the critical problem in seismic response, and thus these degrees of freedom may usually be ignored. The inertia associated with rotational degrees of freedom (Q_7, Q_8, and Q_9) will generally affect lateral mode shapes and their associated frequencies only for tall structures. Most structures of six stories or less will not be tall enough to consider the inertia of these overall floor rotations. The result is a three-degree-of-freedom system. Note, however, that with respect to structure stiffness, joint rotations in frames or rotations of walls at floor levels may not be ignored. These may affect mode shapes and frequencies.

The equation of motion for the three-degree-of-freedom system would be written

$$[M]\{\ddot{U}\} + [C]\{\dot{U}\} + [K]\{U\} = -[M]\{\ddot{X}_b\} \qquad \text{[Eq C4]}$$

where $[M]$ = mass matrix of structure
$[C]$ = damping matrix of structure
$[K]$ = stiffness matrix of structure
$\{U\}$ = three-member vector of floor displacements relative to the base (function of time)
$\{\dot{U}\}$ = three-member vector of floor velocities relative to the base (a function of time)
$\{\ddot{U}\}$ = three-member vector of floor accelerations relative to the base (a function of time)
$\{\ddot{X}_b\}$ = three-member vector, where each member is equal to the base acceleration (function of time).

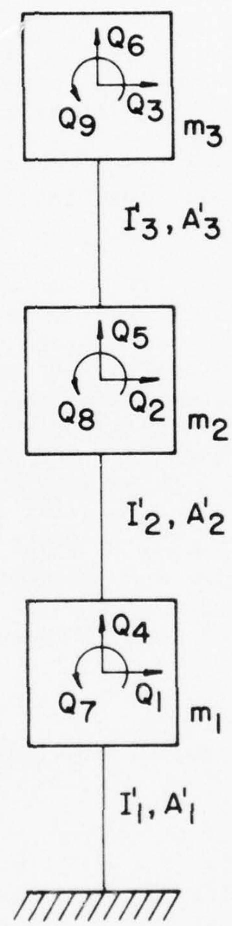

Figure C4. Idealized model for modal analysis.

Eq C4 is similar to the SDF equation, being its "matrix equivalent." The three-degree-of-freedom system is linearly elastic with viscous damping.

The following paragraphs provide some general background in the use of modal analysis for computing the response of a multi-degree-of-freedom system, using a series of SDF systems. The vector, $\{U\}$, is really a list of floor displacements defining a deflected shape of the system. The first variable in the list might represent the response of the first floor, the second variable the second floor, and the third variable the third floor. $\{\dot{U}\}$ and $\{\ddot{U}\}$ are defined similarly. A primary basis of the modal analysis procedure is that at any time during dynamic response, the deflected shape of the structure (the vector $\{U\}$) is a linear function of certain characteristic deflected shapes of the structure, i.e., mode shapes. In essence,

$$\{U\} = a_1\{\phi_1\} + a_2\{\phi_2\} + \ldots + a_m\{\phi_m\} \qquad \text{[Eq C5]}$$

where a_1 through a_m = constants
$\{\phi_1\}$ through $\{\phi_m\}$ = mode shapes
m = number of degrees of freedom.

Note that Eq C5 is defined for a single instant in time and that the a values change with time. The mode shapes themselves, however, are not functions of time. They are totally defined by the mass properties, stiffness properties, and geometry of the structure. Each mode shape is associated with a certain frequency, and with an SDF system corresponding to that frequency. The result is:

$$a_i = c_i\ \xi_i(t) \qquad \text{[Eq C6]}$$

where a_i = the constant for the i^{th} mode
c_i = participation factor (a constant for all times)
$\xi_i(t)$ = the response of the SDF system associated with the i^{th} mode (a function of time).

The deflected shape of the structure as a function of time is finally given by

$$\{U(t)\} = \sum_{i=1}^{m} c_i\{\phi_i\}\xi_i(t) \qquad \text{[Eq C7]}$$

Similarly, the acceleration at each floor level is given by

$$\{\ddot{U}(t)\} = \sum_{i=1}^{m} c_i\{\phi_i\}\ddot{\xi}_i(t) \qquad \text{[Eq C8]}$$

The mathematics of computing the mode shapes, participation factors, and corresponding frequencies may be found in any standard text on structural dynamics.[90,91] The engineer, however, will have a standard computer program to accomplish this task. He/she need only define the mass and stiffness properties of the structure as input for the program. At this point, the engineer has computed c_i and $\{\phi_i\}$ for each mode via a computer program. The response of the SDF system, $\xi_i(t)$, for each mode remains to be determined; the frequency, f_i, is obtained in the modal analysis, and the damping factor, β_i, is the value for which the design response spectrum is written.

Entering the design response spectrum provided for the site in question for the appropriate value of frequency and damping, the maximum *absolute* acceleration, $\ddot{s}_i$, is obtained for the i^{th} mode. The value required, however, is the maximum acceleration *relative* to the base. Hence

$$(\ddot{\xi}_i)_{max} = \ddot{s}_i - \ddot{x}_b \qquad \text{[Eq C9]}$$

where $(\ddot{\xi}_i)_{max}$ = maximum relative acceleration of SDF system for the i^{th} mode

$\ddot{x}_b$ = maximum base acceleration.

The next step is computing the product, $c_i\{\phi_i\}(\ddot{\xi}_i)_{max}$, for each mode. The acceleration for each mode at each floor is then defined, representing an "acceleration shape" for each mode.

The accelerations for the various modes must now be combined at each floor. Since the maximum responses for the various SDF systems do not occur concurrently, it would be too conservative to add them directly. For problems involving several modes (three or more), the root sum square (RSS) approach is recommended. This involves squaring

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[90] J. M. Biggs, *Introduction to Structural Dynamics* (McGraw-Hill Book Co., 1964).

[91] L. Meirovitch, *Analytical Methods in Vibrations* (The Macmillan Co., 1967).

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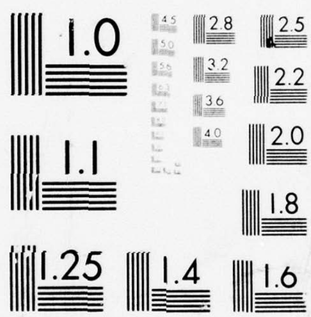

the result for each mode, adding the squares, and calculating the square root of the sum. When only two modes are computed, there is a higher probability that the two modes will attain their maximum responses concurrently. The two modal maxima should be added directly. The maximum relative acceleration is denoted by $\{\ddot{U}\}_{max}$. Finally, this maximum is in the form of an acceleration relative to the base. The ground acceleration should be added to the vector of maximum relative accelerations to obtain an absolute acceleration. Hence,

$$\{\ddot{X}\}_{max} = \{\ddot{U}\}_{max} + \{\ddot{X}_b\} \qquad \text{[Eq C10]}$$

where $\{\ddot{X}\}_{max}$ = three-member vector giving maximum absolute acceleration at each floor level.

$\{\ddot{U}\}_{max}$ and $\{\ddot{X}_b\}$ are as defined previously.

Figure C5 summarizes the modal analysis calculations.

The modal analysis procedure applies only to a linearly elastic system. Inelastic action or ductility is not relevant to this step of the calculations. The results are also specific to the damping factor for which the spectrum has been provided.

Forces in Elements

At this stage of the analysis, the maximum absolute acceleration for each floor is known and is multiplied by the mass of the floor to obtain the applied force at that level for totally linearly elastic response. These forces are applied through the center of mass of the diaphragms to distribute the story loads to various lateral load-resisting elements. The force distribution in each lateral load-resisting element is then computed, preferably by a computer program. Many approximate hand analysis methods, such as the Portal Method, make assumptions that may be unsuitable for this calculation.[92] Among these assumptions are that the point of inflection in a column is at mid-height, the rotations at the top and bottom of a story are equal, and the girders are rigid. A computer program that will account for the compatibility conditions of the structure is necessary to perform these calculations.

[92] R. L. Sharpe, G. Kost, and J. Lord, *Behavior of Structural Systems Under Dynamic Loads*, Building Science Series 46 (National Bureau of Standards, February 1973).

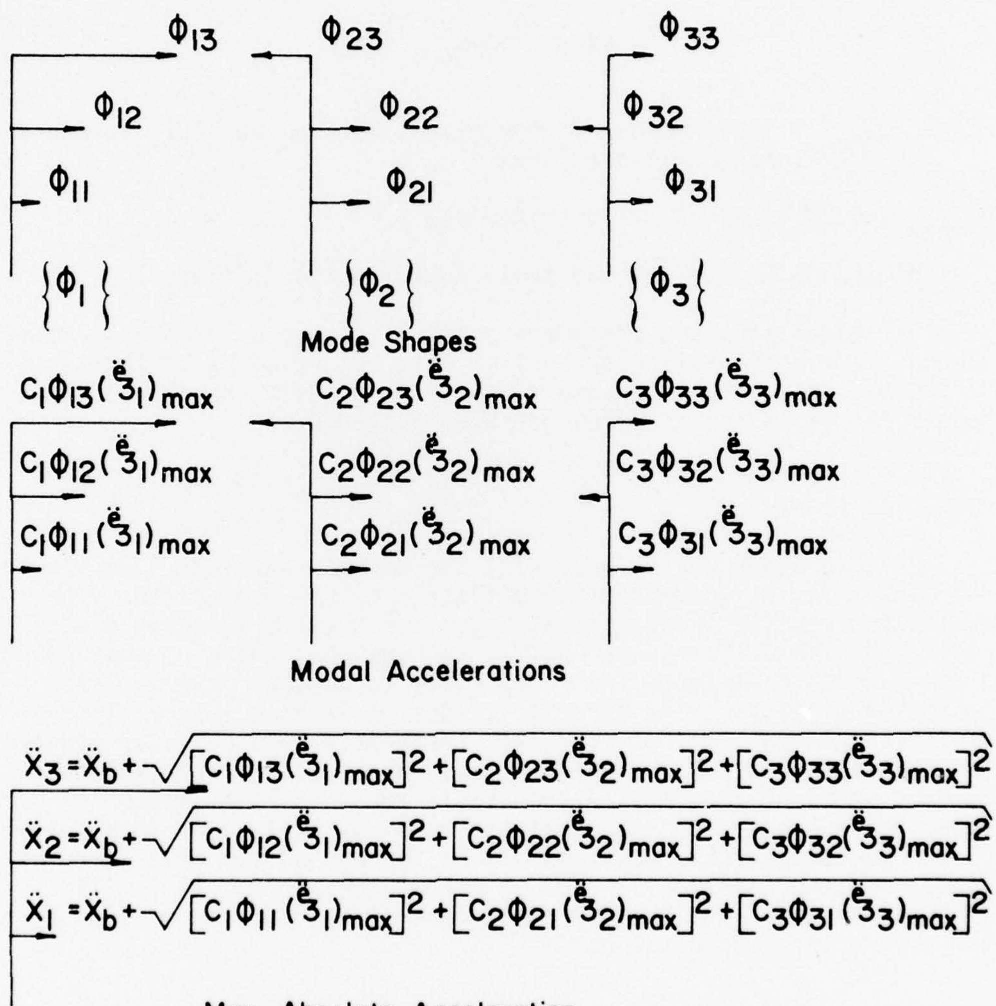

Figure C5. Modal analysis calculations.

Idealization of Structure

The discussion of the previous section involved an idealized model of a structure (Figure C4). The engineer must transform the actual structure into such an idealization before proceeding with the analysis.

The structure is generally assumed to consist of an array of vertical lateral load-resisting elements connected by the floors and the roof, which act as diaphragms (Figure 2). Hence, the lateral load-resisting system has been divided into subsystems, which may consist of several frames, several walls, a coupled wall system, an infilled frame, etc. The mass of the building is often idealized as being concentrated at the diaphragm levels, i.e., at the center of gravity of each floor and the roof.

The stiffness properties of the structure must be provided in order to perform the modal analysis. The form in which these properties are provided largely depends on the type of computer program that will be used to perform the modal analysis. Some programs may require section stiffnesses of individual members; others may require story stiffness, or some other subdivision. The section properties considered for modal analysis are generally those calculcated by assuming the section to be uncracked and transforming steel into concrete through the ratio of their Young's Moduli.

A major question related to modal analysis concerns what degrees of freedom will be used. For most structures of six stories or less, rotations about a horizontal axis need not be considered. These rotations correspond to degrees of freedom 7 through 9 in Figure C4. These generally become important only for tall buildings. Vertical displacements (degrees of freedom 4 through 6) will also generally be ignored. When a building has the potential for torsional response, as described in Chapter 2, the question arises whether this torsion should be considered in the modal analysis. Ignoring vertical motion and rotations about horizontal axes as described above, three degrees of freedom would still be required at each floor level. These would include two horizontal deflections and rotation about a vertical axis. The idealization is shown in Figure C6. The horizontal deflections are u and v, and the rotational degree of freedom is represented by θ. The mass of the j^{th} floor is m_j and r is the rotational inertia about a vertical axis, $\bar{I}_j$. At each story level, two horizontal stiffnesses, K_x and K_y, and a torsional stiffness, T_θ, must be defined. The computation for mode shapes and frequencies would require a three-dimensional structural analysis computer program. Such calculations are expensive and generally impractical. Torsion generally is not considered in the modal analysis stage; it is considered only in the evaluation process. Hence, rotation about a vertical axis is ignored entirely at the

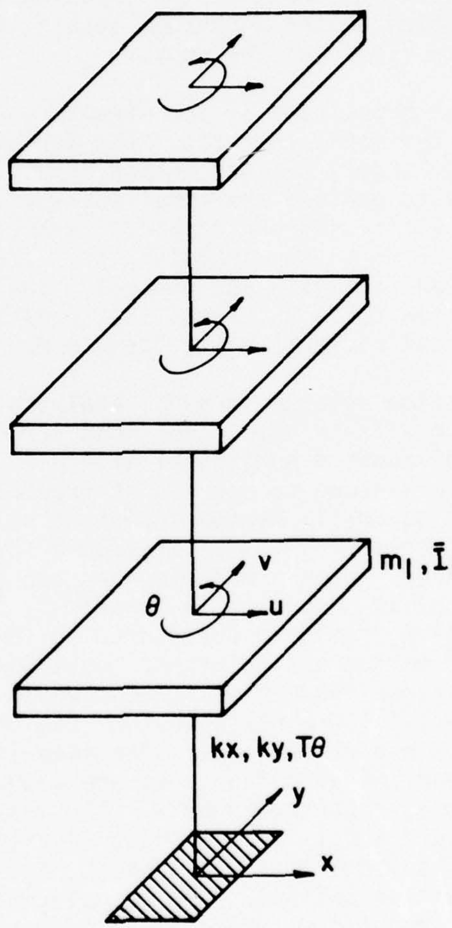

Figure C6. Three-dimensional idealization of structure.

modal analysis stage of the evaluation; if the structure is to be analyzed in both the u and v directions (Figure C6), the analyses are generally performed independently of each other. Consider the structure of Figure 2. If the diaphragm is considered perfectly rigid, modal analysis for x-direction response might use the model of Figure C7, in which the frame and the coupled wall system are connected by a hinged rigid link. If the diaphragm is considered to be perfectly flexible, separate modal analyses would be performed for the frame and the coupled wall system.

Although the structure being analyzed is often considered to be fixed at the base, this assumption may not be valid for certain soil conditions and certain types of foundations. It is possible for the soil and structure to interact dynamically. Such problems, however, are not considered in this report.

Another problem involves the idealization of various wall-frame interaction systems. Coupled wall systems are often best idealized as an equivalent frame (Figure C8). The figure shows the structure on the left, and the equivalent frame idealization on the right. The joint regions (hatched areas) are considered rigid. These rigid blocks are connected by frame members. Since the depths of the columns and beams are almost always large in comparison to the length, shear deformations must be considered. Similarly, axial deformations in the piers, that accompany flexure of the system as a whole, must often be considered. A system consisting of a wall along the same axis as a frame could be idealized similarly, as shown in Figure C9. In this case, axial deformations are probably not considered and shear deformations must be considered only in the pier. Infilled frames can also present idealization problems (Figure C10). The question concerns the degree of composite action between the infill and the surrounding frame. This interaction is sensitive to the quality of the joint. For a perfectly composite case, the moment of inertia may be computed like that for a monolithic section; the system is treated like a structural wall with boundary elements. Considering no composite action would result in the moment of inertia being the sum of the moments of inertia for the infill and two columns. Figure C10 illustrates both cases. The actual behavior of the system is somewhere between the two extremes, and engineering judgment is required. For stiffness calculations, the system may be thought of as a structural wall when a moment of inertia is determined.

When the modal analysis is complete and a set of lateral forces has been computed, element or member forces must be computed. The question of whether story forces, element forces, or member forces should be considered is largely a matter of engineering judgment, but depends greatly on the structure's complexity. If there is an irregular layout, setbacks, or major variations in stiffness, either in plan or height, more attention to localized forces may be necessary.

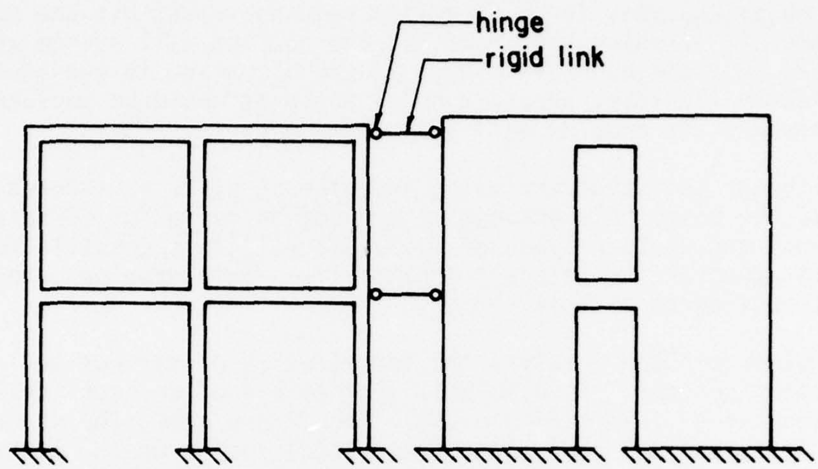

Figure C7. Idealization of diaphragm coupling.

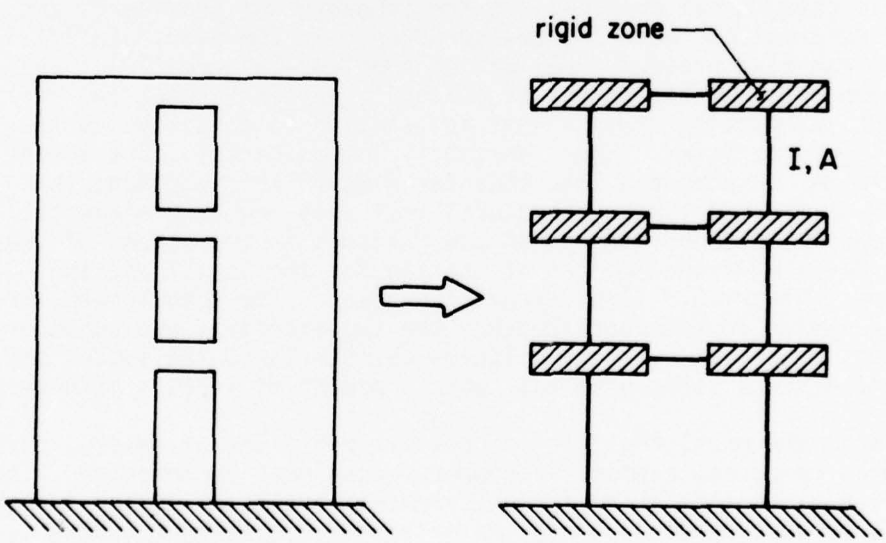

Figure C8. Equivalent frame idealization for coupled shear wall.

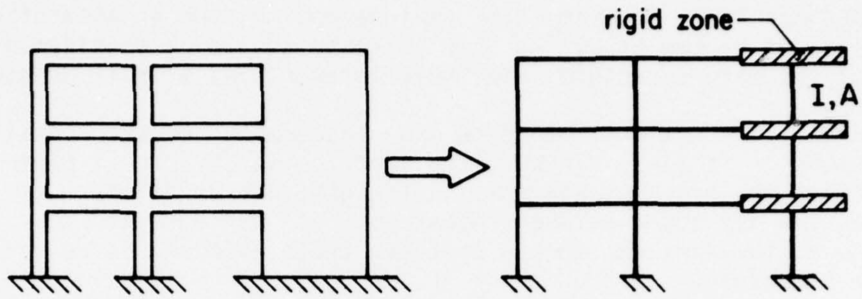

Figure C9. Equivalent frame idealization of wall-frame interaction system.

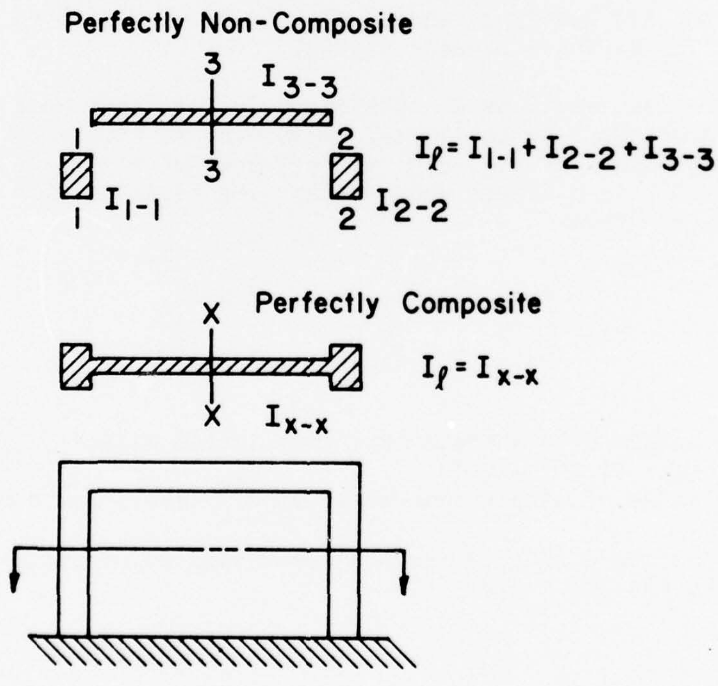

Figure C10. Stiffness for infilled frame.

Modeling appendages or attached structures can be a problem. Considering attached structures to be isolated from the rest of the structure may result in a gross underestimation of forces on the attached portion. An example of failure due to such an assumption is discussed by Lew et al.[93] Such portions should be considered a part of the main structure, and their interactions with it considered.

Finally, a comment should be made concerning "nonstructural" partitions or facades. These often function as structural elements, and affect the system's stiffness, strength, and ductility. If interaction may occur between "nonstructural" elements and the structure, the engineer should consider these elements to be part of the structure.

Approximations for Very Simple Structures

A complete modal calculation may not be warranted for buildings with noncritical functions, a very simple configuration, and fairly high natural frequency. For such structures, a more approximate approach that considers only one mode is more appropriate. Such an option is designed to apply to buildings with natural periods less than 0.35 sec and having a regular plan or outline with no abrupt variations in stiffness between stories.

The approach would be to consider only the first mode and consider the mode shape to be linear. Referring to Figure C5, the absolute magnitudes of ϕ_{11}, ϕ_{12}, and ϕ_{13} are not important, as long as they constitute a linear mode shape. The participation factor, c_1, is computed from

$$c_1 = \frac{m_1\phi_{11} + m_2\phi_{12} + \dots + m_n\phi_{1n}}{m_1\phi_{11}^2 + m_2\phi_{12}^2 + \dots + m_n\phi_{1n}^2} \qquad \text{[Eq C11]}$$

n = number of diaphragms or concentrated masses
m_n = mass of n^{th} level
ϕ_{ij} = value of linear mode shape at n^{th} level, as in Figure C5.

Note that the above formula assumes the masses to be concentrated at the floor levels and roof.

[93] H. S. Lew, E. V. Leyendecker, and R. D. Dikkers, *Engineering Aspects of the 1971 San Fernando Earthquake*, Building Science Series 40 (National Bureau of Standards, December 1971).

An approximate formula may be used to estimate the natural period of the structure, T_1, from the number of stories, n:

T_1 = 0.1 n, for reinforced concrete frames
T_1 = 0.05 n, for buildings with 50 percent or more of their base shear carried by walls
T_1 = 0.05 n, for masonry

Frequency, f_1, is given by $f_1 = \frac{1}{T}$.

It is now possible to apply these approximated values, along with a design response spectrum, to compute the element forces by the same means used to calculate the element forces in the direct modal analysis procedure.

Capacity of Structure

A set of responses of the structure to the design spectrum have now been computed. These assume linearly elastic response. The structure, however, is not expected to respond linearly elastically; rather, it is expected to yield. It is necessary to estimate that yield capacity if the required energy dissipation factor is to be estimated.

The next step, therefore, is to determine the system's yield mechanism. Yield mechanisms for several basic systems were discussed in Chapter 2. As the structural system becomes more complicated, the yield mechanism becomes more complex. The engineer must exercise considerable care in investigating *all* possible mechanisms. Since the correct mechanism is the one producing the lowest yield load, ignoring a possible mechanism may lead to an unconservative result.

Generally, each lateral load-resisting system would be considered separately, each loaded by a set of floor and roof loads which are in the same ratio to each other as the loads determined in the analysis for linearly elastic response; however, this formula is not totally correct. As various lateral load-resisting elements begin to yield, the diaphragm system will facilitate a redistribution of loads from the yielding elements to the less damaged elements. Hence, a yielding element will not experience that initial load distribution, especially if it is one of the later elements to yield. Hopefully, however, the formula will provide a resonable approximation. Based on this assumed load ratio, a magnitude of loading can be determined for any mechanism. (Note that this is a problem in determinant structural analysis.)

The engineer should be careful to employ yield strengths rather than ultimate strengths. As a collapse mechanism forms, the deformations required in one locale to allow the structure to attain an ultimate section strength in another locale may be unreasonably high. Severe compatibility problems can develop; however, there is less chance of this occurring when the member strengths are based on yield. Yield strengths may be computed by the ACI Code, applying the appropriate strength reduction factor.

A final caution concerns the failure mechanism of the structure as a whole, as opposed to the failure mechanisms of various individual lateral load-resisting elements. For example, a wall may have to deform beyond its capacity if an accompanying frame is to form a yield mechanism, or even carry significant loads. In such a case, an analysis of the structure minus its very stiff element might be necessary in addition to an analysis of the entire structure. Similar problems may develop when lateral load-resisting elements are situated at high eccentricities relative to the center of stiffness of the lateral load-resisting system.

Energy Dissipation Factor

The linearly elastic forces in the structure were obtained in the modal analysis. The structure capacity was estimated from the yield mechanism. The energy dissipation factor, α, is obtained by dividing the linearly elastic loads by the structure capacity. In many cases, this calculation would be carried out with respect to base shear only. In essence, one value of α would be obtained, upon which the structure would be evaluated. This would be given by

$$\alpha = \frac{V_e}{V_y} \qquad \text{[Eq C12]}$$

where V_e = base shear computed from linearly elastic behavior and modal analysis

V_y = base shear associated with collapse mechanism.

In some cases, it may be reasonable to compute energy dissipation factors for individual story shears. For story shear,

$$\alpha_{\ell j} = \frac{V_{e\ell j}}{V_{y\ell j}} \qquad \text{[Eq C13]}$$

where $\alpha_{\ell j}$ = energy dissipation factor for the j^{th} story of ℓ^{th} element

$V_{e\ell j}$ = shear in j^{th} story of ℓ^{th} element from linearly elastic behavior and modal analysis

$V_{y\ell j}$ = shear in j^{th} story of ℓth element associated with yield mechanism.

Generally, the probable accuracy of the method is such that it is not reasonable to analyze on a more detailed level than story shears. It must be realized that when an element in the system yields, the distribution of stiffness in the structure is altered. This may change the mode shapes somewhat, but more important, it alters the distribution of linearly elastic forces in the members for any given set of lateral loads. Hence, it alters the distribution of energy dissipation factors computed locally throughout the system. The level of development of the method is such that extremely localized values of computed energy dissipation factors may not be reliable. Even values computed for individual stories may be thought of as guides to where problems might occur in the structure.

The most direct means of translating these energy dissipation factors into required quantities of damping and ductility is through guidelines given by Newmark and Hall.[94-96] Guidelines pertaining to damping are also provided in Hays et al.[97] The guidelines are provided in terms of fraction of critical damping, β, and ductility factor, μ. The fundamental concepts associated with these terms are discussed in Appendix A.

Ductility guidelines are given in terms of the natural frequency range in which the structure resides. The response of an elasto-plastic SDF system experiencing yielding (Appendix A) is compared to that of a totally elastic SDF system. The ratios of the two systems' accelerations and displacements for various values of ductility are considered. Acceleration, of course, is indicative of force.

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[94] N. M. Newmark and W. J. Hall, *Procedures and Criteria for Earthquake Resistant Design*, Building Science Series 46 (National Bureau of Standards, February 1973).

[95] N. M. Newmark and W. J. Hall, "A Rational Approach to Seismic Design Standards for Structures," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 2 (1974).

[96] N. M. Newmark, "Current Trends in the Seismic Analysis and Design of High-Rise Structures," *Earthquake Engineering*, R. L. Wiegel, Coordinating Editor (Prentice-Hall, Inc., 1970).

[97] W. W. Hays et al., *Guidelines for Developing Design Earthquake Response Spectra*, Technical Report M-114/ADA012728 (CERL, June 1975).

The fundamental concepts are that for very stiff structures, the two systems have the same acceleration; for very flexible structures, the two systems have the same total displacement; and for structures of intermediate stiffness, the two systems have the same absorbed energy (see Figure C11). The rules are summarized in Table C1. Most buildings will be in the category of intermediate stiffness.

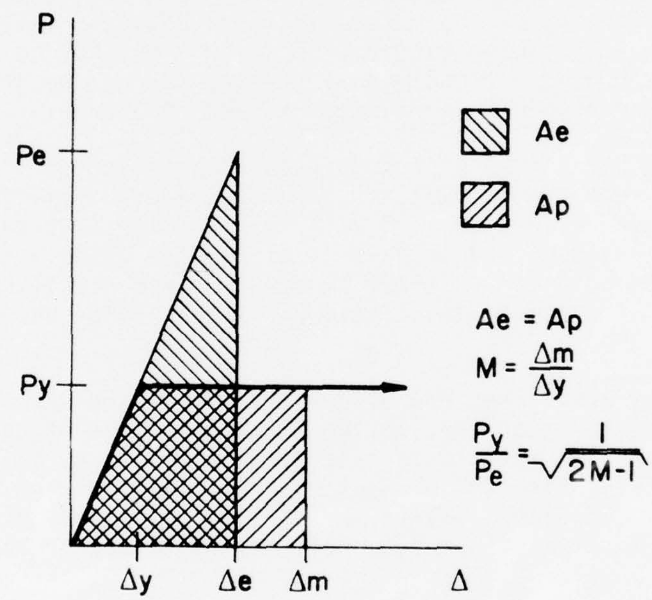

Figure C11. Conservation of absorbed energy.

Table C2 provides damping recommendations. The table lists approximate values of amplification factors of acceleration, displacement, and velocity for linear response spectra characterized by various quantities of viscous damping. The amplification factor is to be multiplied by the maximum base response, or ground motion, to obtain the spectral response. For example, given a maximum base acceleration, $\ddot{x}_b$, for a damping factor, β_s, of 0.02, the table gives a spectral acceleration, $\ddot{s}$, of 4.3 $\ddot{x}_b$. Given a damping factor, β_s, for which the design response spectrum has been developed, the factor

Table C1

Comparison of Elastoplastic and Elastic SDF Systems
(Same Damping Factors)*

Applicable Structure Frequency Range	Quantity Conserved	$\eta = \frac{\text{Elastic Response}}{\text{Elastoplastic Response}}$ Total Displacement	Acceleration
$f < 0.3$ Hertz	Displacement	1	μ
0.3 Hertz $< f <$ 2 Hertz	Displacement very nearly conserved	≈ 1	$\approx \mu$
2 Hertz $< f <$ 6 Hertz	Energy	$\frac{\sqrt{2\mu-1}}{\mu}$	$\sqrt{2\mu-1}$
20 Hertz $< f$	Acceleration	$\frac{1}{\mu}$	1

*From N. M. Newmark and W. J. Hall, *Procedures and Criteria for Earthquake Resistant Design*, Building Science Series 46 (National Bureau of Standards, February 1973); and N. M. Newmark, "Current Trends in the Seismic Analysis and Design of High-Rise Structures," *Earthquake Engineering*, R. L. Wiegel, Coordinating Editor (Prentice-Hall, Inc., 1970), Table 16.1, p 417. Reprinted by permission of Prentice-Hall, Inc., Englewood Cliffs, NJ.

Table C2

Relative Values of Spectrum Amplifications for Various Damping Factors*

Damping as a Fraction of Critical Damping	Amplification Factor for Displacement	Velocity	Acceleration
0	2.5	4.0	6.4
0.5	2.2	3.6	5.8
1	2.0	3.2	5.2
2	1.8	2.8	4.3
7	1.2	1.5	1.9
10	1.1	1.3	1.5
20	1.0	1.1	1.2

*From W. W. Hays, et al., *Guidelines for Developing Design Earthquake Response Spectra*, Technical Report M-114/ADA012728 (CERL, June 1975).

by which the values in the design spectrum must be multiplied to obtain the spectral response values for any other damping factor may be determined from Table C2. Referring to Table C2, if the damping factor of the design spectrum is 0.02 and the response for a damping factor of 0.10 is desired, the relation is

$$\ddot{x}_{0.10} = \left(\frac{1.5}{4.3}\right) \ddot{x}_{0.02} \qquad \text{[Eq C14]}$$

where $\ddot{x}_{0.10}$ = acceleration for β = 0.10
$\ddot{x}_{0.02}$ = acceleration for β = 0.02.

The numerator, 1.5, in the equation was obtained from Table C2 for a damping factor 10 percent of critical (β = 0.10). The denominator, 4.3, was obtained from Table C2 for a damping factor of 2 percent of critical (β = 0.02).

The information in Table C2 was used to assemble Tables C3 and C4 where the multiplication factors, γ_a and γ_d, necessary to define the response level for various quantities of damping in the structure are tabulated for various damping factors in the design spectrum.

A formula (Table C1) has been provided to show the effect of ductility on response. Similarly, Table C3 defines the effect of damping on acceleration. The tables are expressed in terms of two variables, η and γ_a.

The energy dissipation factor computed previously will be related to these variables by $\alpha = \eta\gamma_a$. Hence, with α defined, there exists an array of possible values of η and γ_a. Associated with these is an array of ductility and damping factors, μ and β. These represent the ductility and damping factors that would meet the energy dissipation requirements for the structure to survive the earthquake. Figure 26 in the main body of the report is intended to simplify this step. The figure is a series of interaction diagrams providing the array of ductility and damping factors necessary to obtain several values of energy dissipation factors. Note that each table is for a specific value of damping in the design spectrum. Other tables would be derived for other values of design spectrum damping.

A major idealization, inherent in this approach, should be noted. The method is based on rules of thumb for an SDF system; the structure, however, is a multi-degree-of-freedom system. The implication is that all modes have the same energy dissipation factor and the same damping and ductility; this is not generally the case, however.

Table C3

Effect of Structure Damping on Accelerations Computed From Design Spectrum Damping

Damping of Structure as a Fraction of Critical Damping	$\gamma_a = \frac{\text{Elastic Acceleration for Design Spectrum Damping}}{\text{Elastic Acceleration for Structure Damping}}$						
	Damping of Design Response Spectrum as a Fraction of Critical Damping						
	0	0.005	0.01	0.02	0.07	0.10	0.20
0	1	0.91	0.82	0.67	0.30	0.23	0.19
0.005	1.10	1	0.90	0.74	0.33	0.26	0.21
0.01	1.23	1.12	1	0.83	0.37	0.29	0.23
0.02	1.49	1.35	1.21	1	0.44	0.35	0.28
0.07	3.4	3.1	2.7	2.3	1	0.79	0.63
0.10	4.3	3.9	3.5	2.9	1.27	1	0.80
0.20	5.3	4.8	4.3	3.6	1.58	1.25	1

Table C4

Effect of Structure Damping on Displacement Computed From Design Response Spectrum

Damping of Structure as a Fraction of Critical Damping	$\gamma_d = \dfrac{\text{Elastic Acceleration for Design Spectrum Damping}}{\text{Elastic Displacement for Structure Damping}}$						
	Damping of Design Response Spectrum as a Fraction of Critical Damping						
	0	0.005	0.01	0.02	0.07	0.10	0.20
0	1	0.88	0.80	0.72	0.48	0.44	0.40
0.005	1.14	1	0.91	0.82	0.55	0.50	0.45
0.01	1.25	1.10	1	0.90	0.60	0.55	0.50
0.02	1.39	1.22	1.11	1	0.67	0.61	0.56
0.07	2.1	1.83	1.67	1.50	1	0.92	0.83
0.10	2.3	2.0	1.82	1.64	1.09	1	0.91
0.20	2.5	2.2	2.2	1.8	1.2	1.1	1

Deflections

If two parts of a given building are not completely composite, their individual structural responses will be somewhat dissimilar; they will not deflect in phase with each other, and will not deflect with the same amplitudes. The result may be pounding.[98,99] Interstory drift may also be the cause of certain types of nonstructural damage.

Deflections may be computed by applying the linearly elastic loadings determined previously to the structure. To be more conservative, the engineer may use cracked section stiffnesses. A ductility factor is then applied to the result. This factor may be obtained from the recommendations of Tables C1 and C4.

APPENDIX D:

NONSTRUCTURAL CONSIDERATIONS

Many portions of a building other than its structural system can greatly affect its hazard potential or its ability to function after an earthquake. This includes such parts as partition walls, facades, architectural embellishments, glass, ceiling systems, lighting fixtures, heating, ventilation and air conditioning ducts, fire protection equipment, fire protection coverings, plumbing, elevator shafts, stone and tile veneers, and parapets. Sometimes these systems may be "nonstructural" in name only, since they may behave structurally. It is not the intent in the present report to discuss the evaluation of these systems in a comprehensive manner. A more detailed discussion of nonstructural systems may be found in other publications.[100-104]

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[98] Architectural Institute of Japan, *Design Essentials in Earthquake Resistant Buildings*, (Elsevier Publishing Company, 1970).

[99] H. S. Lew, E. V. Leyendecker, and R. D. Dikkers, *Engineering Aspects of the 1971 San Fernando Earthquake*, Building Science Series 40 (National Bureau of Standards, December 1971).

[100] G. M. McCue, A. Moudon-Vernez, G. Kost, and J. R. Benjamin, *Building Enclosure and Finish Systems: Their Interaction With the Primary Structure During Seismic Action* (June 1975).

[101] G. V. Berg and R. D. Hanson, "Engineering Lessons Taught by Earthquakes," *Proceedings Fifth World Conference on Earthquake Engineering*, Vol 1 (1974).

[102] Architectural Institute of Japan.

[103] E. C. Hillman, Jr. and A. E. Mann, "Architectural Approaches to Hazard Mitigation," *Building Practices for Disaster Mitigation*, Building Science Series 46 (National Bureau of Standards, February 1973).

[104] J. M. Ayres and T. Y. Sun, *Criteria for Building Services and Furnishings*, Building Science Series 46 (National Bureau of Standards, February 1973).

Interaction With Structural System

Nonstructural elements that behave structurally can greatly affect the behavior of the remainder of the structure or may lead to the failure of the "nonstructural" element. Partition walls and masonry exterior veneers often behave structurally and can transform a frame into an infilled frame, raising forces in the structural system above those expected. A similar problem may be caused by a partial-height architectural wall in a frame which creates a spandrel wall assembly with its characteristic short column lengths. Attention should be directed toward these situations when idealizing the structure for analysis.

While the "nonstructural" elements create unexpected forces in the structural system, so does the structural system create unexpected forces in the nonstructural element. The partition wall or veneer may not be able to carry these forces. If the element has little or no reinforcement, its failure may be quite brittle, creating a significant life hazard, either inside or outside the building. Attention should also be directed toward the lateral restraint of such elements since they may not be designed to be stable under applied forces in their own plane.

Architectural Features

Damage to architectural features may present a severe safety hazard and even prevent the operation of the facility.[105] If such fixtures are not to be damaged, they must meet two fundamental requirements:

1. They must resist their own inertial force.

2. They must tolerate the imposed deformations of the structural system.

These requirements, as they relate to several classes of architectural fixtures, are discussed below:

1. Parapets. The connection of the parapet to the building must be capable of resisting the bending movement and shear caused by the inertial forces generated by lateral motion of the parapet.

2. Facades. The connections of the facade to the structure must resist the inertial forces of the facade.

[105] H. S. Lew, E. V. Leyendecker, and R. D. Dikkers, *Engineering Aspects of the 1971 San Fernando Earthquake*, Building Science Series 40 (National Bureau of Standards, December 1971).

3. Tile, stone, and other veneers. These must withstand both their own inertial forces and the deformations of the members to which they are attached.

4. Roofing materials. The structural roof system must be capable of resisting the inertial forces developed by the roofing materials. Because of the problem, many gable roofs, especially in older buildings, may require additional bracing in the longitudinal direction.

5. Lighting fixtures. The connections of these fixtures to the structure must either resist the inertial forces of the fixture, or be equipped with a ball joint that enables the fixture to swing. Care must be exercised, however, to insure that the fixture will not hit anything when it swings.

6. Glass. This must be capable of either withstanding the deformations imposed upon the window frame, or it must be isolated from the frame.

7. Ceiling systems. The connections of the system must tolerate the generated inertial forces; also, the system, as a unit, must tolerate imposed building deformations. Suspended metal lath and plaster ceilings on steel channels are generally both strong and stiff. These will usually transfer inertial forces to the structure, and performance will be satisfactory.[106] Problems are more likely to occur with ceilings composed of metal T-bars with drop-in acoustic tile and light fixtures.[107] The suspension system may not be capable of resisting the inertial forces, and the entire system may not be able to tolerate the deformations of the structure in plan. The two publications referenced below provide standards for hung ceilings and perimeter enclosures, respectively.[108,109]

Mechanical Systems

Mechanical systems include plumbing, heating, ventilation, and air conditioning systems, elevators, and any additional machinery

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[106] E. C. Hillman, Jr. and A. E. Mann, "Architectural Approaches to Hazard Mitigation," *Building Practices for Disaster Mitigation*, Building Science Series 46 (National Bureau of Standards, February 1973).

[107] E. C. Hillman, Jr. and A. E. Mann.

[108] J. M. Ayres and T. Y. Sun, *Criteria for Building Services and Furnishings*, Building Science Series 46 (National Bureau of Standards, February 1973).

[109] E. C. Hillman, Jr. and A. E. Mann.

required to perform the special function of the building. These must be able to tolerate both their own inertial force and the imposed structure deformations. Ayres and Sun provide more detailed information about mechanical systems.[110]

APPENDIX E:

TORSIONAL BEHAVIOR

The general concept of the torsional effect of a building is shown in Figure E1. The floor plate moves with the linear accelerations, $\ddot{u}$ and $\ddot{v}$, through its center of mass, and an angular acceleration, $\ddot{\theta}$, about its center of mass. These correspond to forces $m'\ddot{u}$ and $m'\ddot{v}$ and moment $\bar{I}\ddot{\theta}$, respectively. The angular acceleration and accompanying inertia represent a third degree of freedom at the diaphragm or floor level for purposes of modal analysis (see Appendix C). Except for very critical or unusual structures, this angular acceleration about the center of mass is generally ignored, since torsional response complicates the analysis procedure. The more usual case is depicted in Figure E1b. The hatched block shows a lateral load-resisting element, ℓ. (Other lateral load-resisting elements would exist, but are not illustrated in the figure.) The center of stiffness of all load resisting elements would correspond to the point shown in the figure. For the purpose of distributing force to the lateral load-resisting elements, the forces and moment at the center of stiffness equivalent to those applied at the center of gravity must be determined. This set of forces is shown on the second free body on the figure. It is these forces that are distributed among the lateral load-resisting elements. It should be apparent that the moment M, due to the distance between the center of mass and the center of stiffness, is likely to increase the lateral force in element ℓ over and above that computed for linear acceleration only. This may lead to damage concentrated in certain parts of the building.

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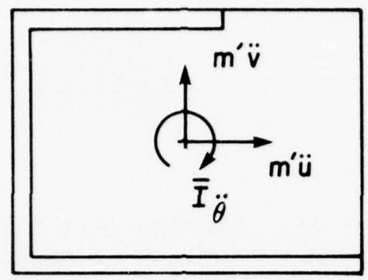
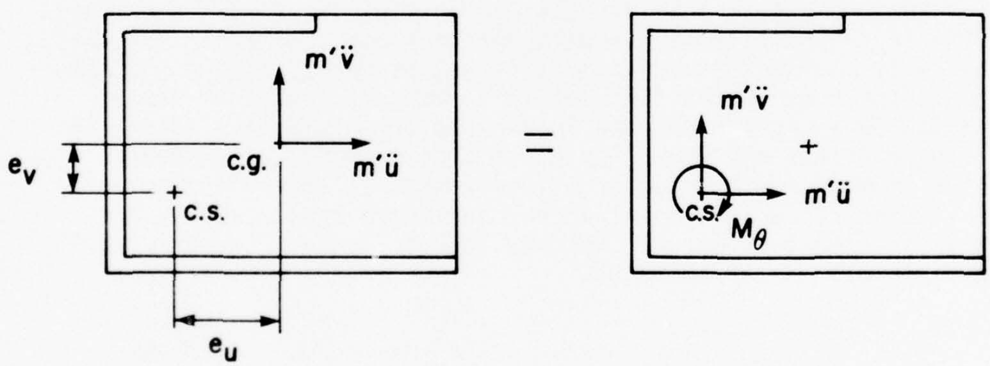
[110] J. M. Ayres and T. Y. Sun, *Criteria for Building Services and Furnishings*, Building Science Series 46 (National Bureau of Standards, February 1973).



(a)

m' = MASS
$\bar{I}$ = ROTARY INERTIA
$\ddot{u}, \ddot{v}$ = LINEAR ACCELERATION
$\ddot{\theta}$ = ANGULAR ACCELERATION



(b)

c.g. = CENTER OF MASS
c.s. = CENTER OF STIFFNESS
M_θ = MOMENT OF $m'\ddot{u}$ AND $m'\ddot{v}$ ABOUT c.s.
ℓ = ℓ^{th} LATERAL LOAD RESISTING ELEMENT

Figure E1. Torsional effects on diaphragm.

APPENDIX F:

SOURCES OF ERROR

Emphasis should be placed on the highly approximate nature of the evaluation procedure. It is a tool to provide data to aid in making a sound judgment. The results should not be followed blindly, since they represent only an approximation. Examples of such approximations are:

1. Variation of actual member section stiffnesses from computed values.

2. Idealizations in member behavior (perfect steel-concrete bond; plane sections remain plane).

3. Effects of joint deformations on stiffness and force distribution.

4. Variability of workmanship.

5. Stress concentrations not fully accounted for.

6. Incomplete interaction in wall-frame interaction-type lateral load-resisting elements.

7. Incomplete consideration of degree of coupling diaphragms provide for various lateral load-resisting elements.

8. Soil-structure interaction.

9. Errors due to lumping mass at floor and roof levels.

10. Probabilistic nature of design response spectrum.

11. Variation of structure response frequency from computed frequency.

12. Idealization as an elastoplastic system.

13. Idealization of multi-degree-of-freedom system as single-degree-of-freedom system for computation of required damping and ductility.

14. Highly approximate nature of rule-of-thumb methods to compute damping and ductility.

15. Computed values of ductility are averaged over entire structure or, at least, a large portion of the structure. There will be localized areas of much higher ductility requirements.

16. Ignoring changes in force distribution that occur as system yields may cause error in computed ductility requirements over the strucuture.

APPENDIX G:

NOTATION

a_b	= acceleration at the base of a structure.
a_i	= a factor (a function of time) characteristic of the i^{th} mode of a structural system.
b	= total length of a rectangular diaphragm.
b_r	= the r^{th} portion of the total diaphragm length, b.
b_w	= width of a reinforced concrete section.
c	= velocity coefficient in a single-degree-of-freedom, viscously damped system.
c_i	= participation factor for the i^{th} mode of a structural system.
$\overline{c}_\ell$	= the distance of the ℓ^{th} lateral load-resisting element from the center of stiffness of a specific diaphragm.
d	= effective depth of the cross-section of a reinforced concrete member or element.
e	= height of compression wedge in failure mechanism for infilled frame.
e_u	= the u-direction component of the distance between the center of mass and center of stiffness of a diaphragm.
e_v	= the v-direction component of the distance between the center of mass and center of stiffness of a diaphragm.
f	= natural frequency of a single-degree-of-freedom system.
f_c	= compressive stress in concrete.
f_i	= natural frequency associated with the i^{th} mode of a multi-degree-of-freedom system.
f'_s	= compressive stress in longitudinal reinforcement.

f_{unc} = natural frequency associated with the unyielded state of an elastoplastic single-degree-of-freedom system.

f_{vy} = yield strength of the shear reinforcement of a reinforced concrete member.

f_y = yield strength of the longitudinal reinforcement of a reinforced concrete member.

h = story height in a building, center to center of diaphragms.

i = subscript to denote a specific mode in a modal analysis.

j = subscript to denote a specific floor or diaphragm level, where $j = 1$ corresponds to the lowest-level diaphragm. The top-level diaphragm is considered to be the roof.

k = spring stiffness in a single-degree-of-freedom system.

k_ℓ = stiffness of the ℓ^{th} lateral load-resisting element to loading at a specific diaphragm level.

k_μ = the reduced stiffness of a single-degree-of-freedom system corresponding to response characterized by a specific ductility, μ.

k_{unc} = stiffness associated with the unyielded state of an elastoplastic single-degree-of-freedom system.

ℓ = subscript to denote a specific lateral load-resisting element.

m = number of modes considered in modal analysis.

m_j = mass associated with the j^{th} diaphragm of a structure.

m' = mass associated with a specific diaphragm of a structure.

m'_ℓ = tributary mass associated with the ℓ^{th} lateral load-resisting element for a specific diaphragm.

m_o = concentrated mass of a single-degree-of-freedom system.

n = total number of diaphragm levels in a structure.

p = longitudinal steel ratio for a reinforced concrete member.

p_b = longitudinal steel ratio for a reinforced concrete member corresponding to simultaneous yielding of tension steel and crushing of concrete.

s_m = maximum center-to-center distance between ties if buckling of longitudinal reinforcement is to be prevented.

s_v = center-to-center spacing of shear reinforcement along the length of a reinforced concrete member.

$\ddot{s}$ = spectral acceleration for a single-degree-of-freedom system.

$\ddot{s}_i$ = spectral acceleration associated with the i^{th} mode of a multi-degree-of-freedom system.

t = thickness of a structural wall.

u = horizontal deflection relative to the ground at a specific diaphragm level.

$\dot{u}$ = horizontal velocity relative to the ground at a specific diaphragm level.

$\ddot{u}$ = horizontal acceleration relative to the ground at a specific diaphragm level.

$\ddot{u}_j$ = maximum acceleration relative to the ground at the level of the j^{th} diaphragm, considering all modes being included in the analysis.

u_ℓ = lateral deflection of the ℓ^{th} lateral load-resisting element at the level of a diaphragm.

v = horizontal deflection relative to the ground in a direction perpendicular to the deflection, u.

$\dot{v}$ = horizontal velocity associated with deflection, v.

$\ddot{v}$ = horizontal acceleration associated with deflection, v.

v_c = shear stress assigned to concrete in a reinforced concrete member.

v_u = total shear stress on the cross section of a reinforced concrete member.

w = width of a rectangular diaphragm.

$\ddot{w}_{ij}$ = maximum acceleration relative to the ground for the i^{th} mode at the level of the j^{th} diaphragm.

$\ddot{x}$ = absolute horizontal acceleration at a specific diaphragm level.

$\ddot{x}_b$ = base acceleration.

$\ddot{x}_j$ = maximum absolute acceleration at the level of the j^{th} diaphragm, considering all modes being included in the analysis.

z = total number of lateral load-resisting elements for a structure.

$\ddot{z}_j$ = maximum absolute acceleration at the level of the j^{th} diaphragm, considering all modes.

A = cross-sectional area of a member.

A_c = area under the load-deflection relation of a single-degree-of-freedom system for linearly elastic response.

A_{c1} = compressive force applied to a joint core by the upper column framing into the joint.

A_{c2} = compressive force applied to a joint core by the lower column framing into the joint.

A_j' = area of the j^{th} story for a planar multi-degree-of-freedom system.

A_p = area under the load-deflection relation of a single-degree-of-freedom system for inelastic response.

A_v = area of shear reinforcement within a length, s_v, along a member.

B = constant equal to approximately 2.2.

C = compressive force in the vertical boundary element of an infilled frame or wall with boundary elements.

C_p = acceleration factor for a nonstructural fixture.

C'' = compressive force in edge beam of diaphragm due to loads in plane of diaphragm.

D = diameter of longitudinal reinforcement in a reinforced concrete member.

E_t = tangent modulus of longitudinal reinforcement where the stress-strain relation is nonlinear and the steel is at the yield stress.

E_w = Young's Modulus for the material in the infill portion of an infilled frame.

F = flexibility factor for diaphragms used in TM 5-809-10. This is equal to the shear flexibility.

F_p = inertial force developed by a nonstructural fixture.

I = moment of inertia of a member.

I_{a-a} = moment of inertia of a reinforced concrete section about the axis a-a where a = 1, 2, 3, ..., x.

I'_j = moment of inertia of the j^{th} story for a planar, multi-degree-of-freedom system.

$\overline{I}_j$ = rotary inertia about a vertical axis for the j^{th} diaphragm.

I_ℓ = total moment of inertia of the ℓ^{th} lateral load-resisting element.

K_x = stiffness of a story in a multi-degree-of-freedom system to bending about y-axis.

K_y = stiffness of a story in a multi-degree-of-freedom system to bending about x-axis.

L = length of flexural member from face of joint to face of joint.

M_{frame} = base moment assigned to the frame portion for an infilled frame or wall with boundary elements.

M_ℓ = moment applied to a diaphragm through the ℓ^{th} lateral load-resisting element.

M' = in-plane moment in a diaphragm generated by lateral forces.

M_u = ultimate section strength of a flexural member, assuming no axial load.

M_{ub1} = ultimate section strength for the beam framing into the left face of the joint core.

M_{ub2} = ultimate section strength for the beam framing into the right face of the joint core.

M_{uc1} = moment applied to a joint core by the upper column framing into the joint.

M_{uc2} = moment applied to a joint core by the lower column framing into a joint.

M_{wall} = base moment associated with the interior portion of an infilled frame or wall with boundary elements.

M_θ = moment about the center of stiffness of a diaphragm generated by inertial forces through the center of mass of the same diaphragm.

P = lateral load applied at the level of a diaphragm.

P_e = maximum force response of a single-degree-of-freedom system considering linearly elastic response.

P_j = lateral load applied to a structure at the level of the j^{th} diaphragm.

P_y = force corresponding to yield of an elastoplastic single-degree-of-freedom system.

Q_r = displacement corresponding to r^{th} degree of freedom in a multi-degree-of-freedom system.

R'_ℓ = area of a specific diaphragm tributary to the ℓ^{th} lateral load-resisting element.

T = tensile force in the vertical boundary element of an infilled frame or wall with boundary elements.

T_i = natural period associated with the i^{th} mode of a multi-degree-of-freedom system.

T' = total tension in the complete cross section of a diaphragm.

T'' = tension force in edge beam of diaphragm due to loads in plane of diaphragm.

T_θ = torsional stiffness to moment about a vertical axis for a story of a multi-degree-of-freedom system.

V = shear force in a structural member.

V_ℓ = shear force in the ℓ^{th} lateral load-resisting element at a diaphragm level.

V_e = total base shear for a structure, based on linearly elastic response.

$V_{e\ell}$ = base shear for the ℓ^{th} lateral load-resisting element in a structure, based on linearly elastic response.

$V_{e\ell j}$ = for linearly elastic structural response, the shear in the j^{th} story of the ℓ^{th} lateral load-resisting element.

V' = in-plane shear force in a diaphragm.

V_u = shear force at the ends of a flexural member, taken at the faces of the joint cores, consistent with the development of the ultimate section moment, M_u, at the faces of the joint cores.

V_{ub1} = shear force applied to a joint core from the beam framing into the joint from the left, consistent with the development of the ultimate section moment, M_{ub1}, at the joint face.

V_{ub2} = shear force applied to a joint core from the beam framing into the joint from the right, consistent with the development of the ultimate section moment, M_{ub2}, at the joint face.

V_{uc1} = shear force applied to a joint core from the column framing into the joint from the top, consistent with the development of the ultimate section moment, M_{uc1}, at the joint face.

V_{uc2} = shear force applied to a joint core from the column framing into the joint from the bottom, consistent with the development of the ultimate section moment, M_{uc2}, at the joint face.

V_y = total base shear for a structure, corresponding to a collapse mechanism.

$V_{y\ell}$ = base shear for the ℓ^{th} lateral load-resisting element in a structure, corresponding to a collapse mechanism.

$V_{y\ell j}$ = shear force in the j^{th} story of the ℓ^{th} lateral load-resisting element, corresponding to a collapse mechanism.

W_p = weight of a nonstructural fixture.

α = energy dissipation factor for the entire structure.

α_ℓ = energy dissipation factor for the ℓ^{th} lateral load-resisting element.

$\alpha_{\ell j}$ = energy dissipation factor for the j^{th} story of the ℓ^{th} lateral load-resisting element.

α_o = energy dissipation factor for a single-degree-of-freedom system.

β = required damping capacity of a structure or element.

β_s = damping factor for which the design spectrum has been developed.

γ_a = ratio of the linearly elastic acceleration response of a structure computed for the damping factor inherent in the design response spectrum to the linearly elastic acceleration response computed for the damping factor of the structure.

γ_d = ratio of the linearly elastic displacement response of a single-degree-of-freedom system computed for the damping factor inherent in the design response spectrum to the linearly elastic displacement response computed for the damping factor of the system.

γ_o = ratio of the linearly elastic acceleration response of a single-degree-of-freedom system computed for the damping factor inherent in the design response spectrum to linearly elastic acceleration response computed for the damping factor of the system.

Δ = lateral deflection of a single-degree-of-freedom system.

Δ_{ej} = interstory drift in the j^{th} story computed from linearly elastic assumptions and design response spectrum damping.

Δ_j = interstory drift at the j^{th} story adjusted for structure damping and ductility.

Δ_m = maximum lateral deflection of a single-degree-of-freedom system.

Δ_y = lateral deflection of a single-degree-of-freedom system corresponding to yield.

η = for a given damping factor, the ratio of acceleration response of a structure, considering linearly elastic response, to the acceleration response of the structure, considering elastoplastic behavior.

η_o = for a given damping factor and single-degree-of-freedom system, the ratio of acceleration response considering linearly elastic behavior to the acceleration response for elastoplastic behavior.

θ = rotation of a diaphragm about a vertical axis.

$\ddot{\theta}$ = acceleration of a diaphragm about a vertical axis.

μ = required ductility of structure or element.

ξ_i = displacement relative to the ground for the single-degree-of-freedom system corresponding to the i^{th} mode.

$\ddot{\xi}_i$ = acceleration relative to the ground for the single-degree-of-freedom system corresponding to the i^{th} mode.

$(\ddot{\xi}_i)_{max}$ = maximum acceleration relative to the ground for the single-degree-of-freedom system corresponding to the i^{th} mode.

σ = uniform bearing stress used to evaluate stability of compression wedge of failure mechanism for infilled frame.

$\{\phi_i\}$	=	vector consisting of the shape of i^{th} mode for a multi-degree-of-freedom system.
ϕ_{ij}	=	mode shape value for the i^{th} mode at the j^{th} diaphragm level.
$[C]$	=	viscous damping matrix for a multi-degree-of-freedom system.
$[K]$	=	stiffness matrix for a multi-degree-of-freedom system.
$[M]$	=	mass matrix for a multi-degree-of-freedom system.
$\{U\}$	=	vector of displacements relative to the base for a multi-degree-of-freedom system.
$\{\dot{U}\}$	=	vector of velocities relative to the base for a multi-degree-of-freedom system.
$\{\ddot{U}\}$	=	vector of accelerations relative to the base for a multi-degree-of-freedom system.
$\{\ddot{U}\}_{max}$	=	vector of maximum accelerations relative to the ground for a multi-degree-of-freedom system. This vector generally contains the maximum accelerations at each diaphragm level relative to the ground.
$\{\ddot{X}_b\}$	=	vector of length equal to the number of degrees of freedom, where each member of the vector is equal to the base acceleration.
$\{\ddot{X}\}_{max}$	=	vector of maximum absolute accelerations for a multi-degree-of-freedom system. This vector generally contains the maximum absolute accelerations at each diaphragm level.

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